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TRANSACTIONS

OF THE

ASSOCIATION

OF

CIVIL ENGINEERS

OF

CORNELL UNIVERSITY

VOL. VI 1897–1898

CONTAINING

ADDRESSES BY NON-RESIDENT LECTURERS, MISCELLANEOUS
PAPERS, CONSTITUTION, AND LIST OF MEMBERS
OF THE ASSOCIATION

NOTE.—This Association is not responsible for any statements or opinions advanced in any of its publications.

ITHACA, NEW YORK
JUNE, 1898

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REMOTE STORAGE

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S1894

CONSTITUTION

OF THE

ASSOCIATION OF CIVIL ENGINEERS

OF CORNELL UNIVERSITY.

PREAMBLE.

We, the undersigned, members of the Senior and Junior classes in the College of Civil Engineering of Cornell University, do hereby form ourselves into an Association for the discussion of engineering topics, and the promotion of general information on engineering subjects, and do hereby agree to abide by, and sustain the following Constitution and By Laws:

ARTICLE I.

NAME.

1. This Association shall be known as the Association of Civil Engineers of Cornell University.

ARTICLE II.

MEMBERSHIP.

1. The Association shall consist of Active and Honorary members.

2. All Alumni of this college and all students recognized as upperclassmen, and registered in the College of Civil Engineering, are eligible to membership in this Association.

3. Any eligible person may become an honorary member by a two-thirds vote of the members present at any regular meeting. Such members shall have privileges of active members except those of voting and holding office, and shall be exempt from all dues.

4. The membership fees for this Association for all active graduate members shall be $1.00 per annum. All money received from such membership fees shall be devoted to defraying cost of publication of non-resident lectures delivered before the Association. All other expenses of this Association shall be met by direct tax upon the undergraduate members.

5. A copy of each lecture delivered before this Association shall be forwarded to each member of the Association.

ARTICLE III.

OFFICERS.

1. The officers of the Association shall consist of a President, Vice-President, Recording Secretary, Corresponding Secretary, and Treasurer.

2. The President shall preside at all meetings of the Association and enforce the Constitution and By-Laws, and shall call special meetings at the request of five active members.

3. The Vice-President shall take the chair at the request of the President, and shall act as President in his absence. The Vice-President shall be chairman of the appointment committee.

4. The Recording Secretary shall keep minutes of the proceedings of all meetings of the Association and shall post notices for the same.

5. The Corresponding Secretary shall attend to all the necessary correspondence of the Association. He shall be elected from the Faculty of the college.

6. The Treasurer shall receive all money and dues, and shall pay all bills of the Association, such bills to meet the approval of the Executive Committee before such payments. He shall make a report when called upon by the Association and also when his term of office expires. He shall be chairman of the Executive Committee.

7. The officers shall be chosen by ballot at the last regular meeting of the spring term, from the Junior Class, and shall hold office until their successors are elected.

8. All vacancies shall be filled by a special election—only members of the Senior Class being eligible to hold office.

ARTICLE IV.

COMMITTEES.

1. There shall be three Standing Committees, an Executive Committee, a Committee on Appointments, each consisting of three members, and a Publication Committee, consisting of five members. These committees shall be appointed by the President at the beginning of the year.

2. The Executive Committee shall see that the rooms of the Association are ready for occupancy previous to all meetings, and shall transact such business as may be referred to it by the Association

3. The Committee on Appointments shall make appointments for all literary exercises for each meeting, and such appointments shall be posted at least two days before the meeting. The committee shall furnish the Secretary with a list of such appointments.

4. The Publication Committee shall have charge of the compiling, publication, and sale of the Transactions.

ARTICLE V.

AMENDMENTS.

The Constitution or By-Laws may be amended by a two-thirds vote of all members present at any regular meeting; such amendment to be before the Association at least one week.

AMENDMENT I.

The initiation fee for membership to this Society shall be one dollar, and this fee shall cover all dues for the first year of membership. For each succeeding year, an annual due of one dollar shall be charged. The annual membership fee shall entitle the member to a copy of the Transactions published during such year of membership.

BY-LAWS.

ARTICLE I.

REGULAR MEETINGS.

Regular meetings shall be held once in two weeks, in the Association rooms, commencing on the first Friday after registration week, and ending on the last Friday but one before examination week of each term.

ARTICLE II.

QUORUM.

One-third of the active undergraduate members of the Association shall constitute a quorum. No business can be transacted without a quorum being present.

ARTICLE III.

ORDER OF PROCEEDINGS AT A REGULAR MEETING.

1. Roll call.
2. Minutes of Preceding Meeting.
3 Literary Exercises.
4. Unfinished Business.
    a. Report of Standing Committees
    b. Report of Special Committees.
    c. Report of Officers.
    d. Miscellaneous Business
5. New Business.
6. Adjournment.

ARTICLE IV.

EXERCISES.

The exercises shall consist of discussions, memoirs, essays, papers, lectures, and such other exercises as the Association shall from time to time direct.

ARTICLE V.

SUSPENSION OF BY-LAWS.

A By-Law may be suspended for one meeting by a vote of two-thirds of the members present.

H. R. LORDLY,
E. J. FORT,
H. D. ALEXANDER,
Committee.

OFFICERS FOR 1897-98.

President,

HARLEY S. GIBBS.

Vice-President,

F. D. RHODES.

Treasurer,

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Corresponding Secretary,

PROF. CHAS. L. CRANDALL.

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Publication Committee,

C. S. MOORE, *Chairman,*
O. A. WAIT
EDGAR JOHNSTON,
ALFRED FRANK,
WAGER FISHER.

PRESIDENT'S ADDRESS.

Members of the Association of Civil Engineers of Cornell University:

To-day we stand at the close of the fifth year since the re-organization of the Association, which we may safely say is now permanently established. With the assurance that we have back of us the large number of graduate members amounting to over four hundred, with their loyalty to their College and Association, we cannot help but feel that as the Association grows older in years and experience it will become of more and more benefit to both graduate and undergraduate members.

A brief review of the work done for the past year may not be uninteresting.

Meetings have been held every two weeks throughout the year. The programmes have consisted of lectures by non-resident engineers of special ability in their particular branches of the profession, of lectures by members of the faculty and instructing body of the College, and of the usual addresses, papers and discussions by the members of the Association. At one meeting, after listening to a very interesting talk by Professor Church, upon the construction of the new hydraulic laboratory, the body adjourned to the site of operations to view the work just described. Upon another occasion the Association took an excursion to Athens and Sayre, Pa., where, under the direction of Professor Jacoby the works of the Union Bridge Co , and the car and locomotive shops of the Lehigh Valley R. R. were inspected. Of special interest among the lectures given by the faculty may be mentioned that by Professor Fuertes, the Director of the College upon "The Sanitation of Santos," that by Mr. Ogden on "The Ithaca Sewer System," and that by Professor Jacoby on "The Filing of Clippings, Circulars, Manuscript Notes, and References to Articles in Periodicals," which, though given before some three years ago was repeated by request. We highly appreciate the interest which the members of the faculty take in the meetings, for which I extend them our sincere thanks.

Following the advice of our last year's president, the constitution has been slightly revised so as to correspond with our present needs.

It gives me great pleasure to state that though we have been in debt for a number of years, the amount being $60 three months ago, owing to the efforts of our Corresponding Secretary to collect back dues from the graduate members and to the hearty response which he met with; this has been entirely cleared off, leaving us with a balance in the treasury.

I would take this opportunity to say to the graduate members that if we are to continue to publish the Transactions we must have your support. It seems to be asking a very little from each one, and yet upon it depends the future existence of our publication, the merit of which you all recognize. We most cordially thank those of you who have been so kind as to contribute valuable articles to the Association, a number of which will be found in Vol. VI of the Transactions.

Our books show an undergraduate membership of 43, besides 4 graduates taking work in the College who are in regular attendance. Of these I regret to say that the average attendance at meetings, not non-resident lectures, has been but about 20.

Why is it, that in glancing over the addresses of the past presidents, we find in nearly every instance that an urgent appeal is made to keep up the interest in the meetings, which is likewise repeated by the retiring president? It seems to indicate two things, 1st, that the benefit to be derived from these meetings is worth more than the time and trouble spent in their attendance, and 2nd, that there is a tendency on the part of the members to fail to fully appreciate this benefit. Besides the knowledge acquired upon engineering subjects by those who merely listen, there is (and this is of more importance) the experience gained by those who take an active part. It is particularly this latter point which we now fail to appreciate, but will realize it later when the opportunity has passed. As our Director has often said, though an engineer may fully understand his subject, unless he is able to convince others that he does understand it, he will never be truly successful. Taking part in the meetings of the Association is an excellent method for developing this ability.

I would hence advise that the appointment committee make a special effort to have as many as possible of the members contribute to the programmes. As soon as possible at the beginning of the term make out a program for the whole term, giving all ample time for preparation. Attendance at and participation in the meetings should be made a privilege and a pleasure, and not alone a duty. Begin the meetings promptly on time and have them brief but interesting. I would recommend that one or two excursions be taken during the year to some nearby plant or work of engineering interest. In this way all the members are brought together in a social and informal way, which cannot but strengthen the interest in the Association, to say nothing of the methods of engineering practice learned.

On behalf of the Association I wish to thank Professor Crandall who for years has contributed largely to the success of the Transactions, and as Corresponding Secretary has worked faithfully for our interests.

To the committees on Publication and Appointments together with the other officers of the Association we owe much for the manner in which they have performed their duties, which in most cases involve the expenditure of much time and labor.

Thanking you for the support which you have given to the President and to the Assoctation for the past year, and with best wishes for its future success in all its lines of work, I must bid you farewell,

Harley S. Gibbs.

May 13th, 1898.

OFFICERS FOR 1898-99.

President,

FRIEND P. WILLIAMS.

Vice-President,

L. H. McHARG.

Treasurer,

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Corresponding Secretary,

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Recording Secretary,

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Publication Committee,

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E. E. LANPHER,
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W. O. WHITE.

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CORNELL UNIVERSITY.

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Higley, A. H.
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Lueder, A. B.
McHarg, Lester.
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Mildon, R. B.
Moore, E. J.
Reyna, S.
Rue, M. A.
Thompson, A., Jr.
Trautwine, J. C., 3d.
Williams, F. P.
Young, H. A.

MEMBERS OF

THE ASSOCIATION OF CIVIL ENGINEERS,

CORNELL UNIVERSITY.

ALL GRADUATES OF THE COLLEGE OF CIVIL ENGINEERING UP TO AND INCLUDING 1898.

The address on the line with the name is the home or permanent one; that on the line below is the business or usually more temporary one. When only one is given its position is an index of its permanency, except when forced from one position to the other by lack of room. All the addresses have been verified since March 1, 1898, except those in italics, which are unreliable. All changes in address should be reported promptly to the Corresponding Secretary.

Agate, Elroy T. . . . . *C.E.*, '97 . . . . Pittsford, N. Y.
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Allen, Charles F. . . . . *C.E.*, '73 . . . . 2610 Lafayette St., Denver, Colo.
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Atwood, William G. . . . . . *C.E.*, '92 . Rampart City, Minnook Creek, Alaska.
Avery, Fred H. . . . . *C.E.*, '97 . . . . 421 Superior St., Chicago, Ill.
Asst., C. P. R.R., British Columbia, Canada.
Aylen, John . . . . *C.E.*, '77; M. Can. Soc. C. E.; Aylmer, Canada.
Civil Engr., with Me. & N. H. Gran. Const. Co. of Portland, Me.
Ayres, C. Morton . . . . *C.E.*, '94; M. Mich. Ornithological Club.
Recdr., Improving St. Mary's River, U. S. Engr's. Office,
Sault Ste. Marie, Mich.
Bacon, Geo. Morgan . . *C.E.*, '93 . . . . West Medford, Mass.
Transitman, Boston Transit Com., 20 Beacon St., Boston, Mass
Bailey, Earl B. . . . . *C.E.*, '94 . . . . E. Aurora, N. Y.
Trav. Salesman, The G. N. Pierce Co., 566 Main St., Buffalo, N Y.
Baker, Chas. H. . . . . *C.E*, '86 . . . . Seattle, Wash.
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Balcom, H. Gage . . . . . *C.E.*, '97 . . . . . Bergen, Genesee Co., N. Y.
Draftsman, Berlin Iron Bridge Co., East Berlin, Conn.

Baldwin, Ernest H. . . . . . *C.E.*, '92 . . . . 23 Prospect St., Clinton, Mass.
With Met. Water Board.

Banks, John E. . . . . *C.E.*, '92 . . . . Knoxville, Ia.
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Barbour, Irwin W. . . . . . *C.E.*, '94 . . . . Woodfords, Me.
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Transitman, Metropolitan Water Board, Clinton, Mass.
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Prof. Civil Engineering, Cornell University, Ithaca, N. Y.

Crane, Albert S. . . . . . *C.E.*, '91; Jun. Am. Soc. C. E., Mem. Bos. Soc. C. E., Mem. Brook. E. Club.
Asst. Engr., Dept. Sewers, Borough of Brooklyn, 50 Municipal Bldg.

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Trtn. Bur. Bd. Pub. Impts., New York. 177th St. & 3rd Ave.

Wait, John C. . . . . *C.E.*, '82, *M.C.E.*, '91, *L.L.B.*; M. Am. Soc. C.E.;
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Civil Engineer, Cedar Rapids, Ia.

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Washburn, Frank S. . . *C.E.*, '83; M. Am. Soc. C.E., M. West. Soc. C.E.
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M. E. Cl., Phila.
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Weed, Addison . . . . . . *C.E.*, '79 . . . . New Hartford, N. Y.
Farmer.
Welker, Philip A. . . . *C.E.*, '78; M. Nat. Geog. Soc. . . Washington, D. C.
Asst. U. S. Coast and Geod. Survey.
Wheelock, Charles B. . *C.E.*, '76 . . . . Boston, Mass.
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Whelpley, J. R. . . . . . . *C.E.*, '96 . 800 East Cap. St., Washington, D. C.
Rodman, Sewer Div., D. C. Eng. Dept.
Whiskeman, James P. . . . *C.E.*, '98 . . . . 30 S. 4th St., Reading, Pa.
White, Timothy S. . . . . . . *C.E.*, '73; M. Am. Soc. C. E., M. E. Soc. W. Pa.
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Wick, Richard B. . . . *C.E.*, '81 . . . . Warren, Ohio.
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With American Bridge Works, 40th St. and Stewart Ave., Chicago, Ill.
Works, Norris M. . . . . *C.E.*, '97 . . . . Lima, N. Y.
U. S. Sub. Inspr., Buffalo Breakwater, 1101 Morgan Building.
Zarbell, Elmer. . . . . . *C.E.*, '95 . . . . 4132 Ellis Ave., Chicago, Ill.
Asst. Engr. M. & O. R.R., Montgomery, Ala.

FUERTES GOLD MEDALISTS.

UNDERGRADUATES.

Albert Lloyd Colsten,	1895
William Mackintosh,	1896
Gilbert Powers Ritter,	1897

GRADUATES.

John Fillmore Hayford,	1895
Elon Huntington Hooker,	1896
Erasmus Darwin Preston,	1897

For the conditions of the award reference is made to the University Register.

DECEASED MEMBERS.

NAME.		RESIDENCE.	DATE OF DEATH.
Ames, Willis C	*C.E.*, '77 ;	Whitney's Point, N. Y.,	Feb. 23, 1894
Aylen, Charles P	*C.E.*, '76 ;	Aylmer, Canada,	April, 1894
Bueno, Francisco de A. V	*C.E.*, '76 ;	Rio de Janeiro, Brazil,	About 1881
Carpenter, Frank DeY	*C.E.*, '73 ;	*M.C.E.*, '76 ; Highland, N. Y.,	Dec. 19, 1883
Clark, Ira E	*C.E.*, '72 ;	Weston, Mass.,	May 23, 1882
Cook, Isaac N	*C.E.*, '75 ;	Jersey City, N. J.,	May 7, 1885
Cooper, Edgar H	*C.E.*, '85 ;	New York City,	Oct., 1890
Dodd, Franklin M. G.	*C.E.*, '90 ;	Franklin, N. J.,	Sept. 13, 1891
Dobraluboff, John A	*C.E.*, '74 ;	Nijney Novgorod, Russia,	About 1882
Eidlitz, Alfred F	*C.E.*, '76 ;	New York City,	April 22, 1877
Farnham, Whitfield	*C.E.*, '71 ;	*M.C.E.*, '74 ; St. Louis, Mo.,	April 13, 1895
Ferguson, Nicholas C	*C.E.*, '79 ;	St. Louis, Mo.,	Sept. 22, 1896
Fitch, William R	*C.E.*, '74 ;	Ithaca, N. Y.,	April 14, 1886
Foster, Reuben B	*C.E.*, '74 ;	*M.C.E.*, '77 ; S. Lake Weir, Fla.,	Nov. 7, 1895
Greene, Almon C	*C.E.*, '75 ;	Palmyra, N. Y.,	July 28, 1897
Gunner, Daniel W	*C.E.*, '87 ;	Schagticoke, N. Y.,	Oct. 10, 1887
Holbrook, Ernest M	*C.E.*, '89 ;	*M.C.E.*, '90 ; Ithaca, N. Y.,	Oct. 9, 1892
Hulse, Howard C	*C.E.*, '91 ;	Brooklyn, N. Y.,	Feb. 20, 1893
Landers, Herbert H	*C.E.*, '90 ;	Green Island, N. Y.,	Feb. 4, 1893
Lyman, George F	*C.E.*, '73 ;	Tenafly, N. J.,	Dec. 25, 1880
MacMullen, Justus C	*C.E.*, '76 ;	Unionville, N. Y.,	Jan. 31, 1888
Preston, Kolce	*C.E.*, '73 ;	Wilmington, Del.,	Jan. 4, 1876
Rogers, Jesse A	*C.E.*, '91 ;	Evans Mills, N. Y.,	April 24, 1897
Sheldon, Daniel C	*C.E.*, '83 ;	Delphi, N. Y.,	Oct. 2, 1893
Shepard, Frank W	*C.E.*, '86 ;	Medina, O.,	Feb. 10, 1892
Smith, George La T	*C.E.*, '71 ;	*M.C.E.*, '74 ; Canandaigua, N.Y.,	June 25, 1892
Smith, William J	*C.E.*, '79 ;	Charleston, N. Y.,	Dec. 3, 1886
Stewart, Neil, Jr.	*C.E.*, '87 ;	York, N. Y.,	March 30, 1891
Tilley, George A	*C.E.*, '73 ;	Washington, D. C.,	March 14, 1877
Tompkins, John H	*C.E.*, '73 ;	Poughkeepsie, N. Y.,	July, 1879
Viegas-Muriz, Joaquim	*C.E.*, '77 ;	Piracicaba, Brazil,	About 1883
Wightman, Willard H	*C.E.*, '81 ;	Ashland, Ore.,	Oct. 29, 1889

In Memoriam.

JESSE ALANSON ROGERS.

The many friends of Jesse A Rogers, C. E. '91, will be pained to learn of his sudden death near Evans Mills, N. Y., at 5 p. m. Saturday, April 24, 1897.

As a student, Mr. Rogers was exceptionally faithful and thorough and his development and engineering successes have been brought about by his patient determination. He was a hardworking student and had at all times the respect and good will of his fellows.

For several years after graduation, he was with the Berlin Iron Bridge Co., of East Berlin, Conn., devoting himself to that branch of Engineering which he had chosen for himself.

For the year before his death, he had been employed by the Solvay Process Co., of Syracuse, and a few weeks before his death he had undertaken the management of two farms at his old home.

He was married in 1896 to Miss Burnham, of New Haven, Conn., who, with his three brothers and his father and mother have the sympathy of all his old associates.

His illness was at first a slight attack of pneumonia, developing within a week into the erysipelas which proved fatal.

ALMON C. GREENE, C. E., '75.

The death of Almon C. Greene occurred on July 28 of last year at Copenhagen, Lewis Co., N. Y., at the age of forty-four, after a long and painful illness.

For several years after graduating he was engaged in teaching, but he soon received an appointment to a clerkship in the State Senate at Albany and was afterwards promoted to the position of Junior Clerk, which place he held for five years.

Later his numerous business interests claimed his attention so that he never entered the active practice of his profession.

Some four years ago his health failed and much of his time was afterwards spent in Colorado in the hope that the change of air would stay the progress of the dreaded disease, consumption. About a year ago he returned to Copenhagen and soon began to fail quite rapidly.

He was genial and public spirited and had many friends both at home and throughout the state. Always honorable and upright his death is deeply regretted by all who knew him.

A wife and one child survive him, as also a brother.

A FEW CONSIDERATIONS OF MUNICIPAL WATER SUPPLY, WITH SPECIAL REFERENCE TO THAT OF ROCHESTER, N. Y.[1]

BY E. KUICHLING, C.E.

The subject of the water supply of large communities presents so many matters of interest, that it becomes impossible to review all of them within the limits of a single lecture ; and as considerable time is needed for exhibiting the numerous lantern slides of views taken during the construction of the new conduit of the Rochester, N. Y., Water Works, which will doubtless be more entertaining than any written description of the work, it has been deemed expedient to restrict the introductory remarks to some cursory glances at a few of the principal problems of water supply engineering.

The first of these problems is the determination of the proper quality and quantity of water which should be provided per head of population per day ; and when this has been done, the most expedient source from which it is to be obtained can be considered. With regard to quality, it may briefly be stated the water should be as pure as possible, remembering that absolute purity is practically unattainable. From long experience, certain numerical limits of impurity have been fixed by chemists and biologists, within which a water supply may be considered as good or tolerable, and beyond which it should be rejected without hesitation ; but there is always difficulty in deciding how near to any limit a suspicious water may come and still be used with a reasonable degree of safety. The difficulty, moreover, is greatly increased when the procuring of a supply from a different and more desirable source involves much more expense.

In estimating the quality of drinking water, reference is made to the respective amounts of mineral and organic matter which is found in the source ; and when this work is done in a thorough manner, every effort is made to determine the character and condition of the organic matter. The microscope is a valuable aid in this connection, as it enables us to recognize forms of vegetable and animal life, and also at times the products of their disintegration and decomposition. Careful inspection of the source of the supply and its surroundings is likewise of utmost importance, in order to account intelligently for the presence of the matter which may be found in the laboratory.

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[1] Lecture delivered by E. Kuichling, C.E., Chief Engineer Rochester Water Works, before the College of Civil Engineering of Cornell University.

The mineral or inorganic matter which is contained in the water is generally determined with great facility and accuracy, but it is quite different in the case of organic material. This consists mainly of carbon, hydrogen, nitrogen and oxygen, united in various proportions. Animal matter usually contains much more nitrogen than vegetable matter and decomposes more rapidly. By decomposition, is meant the gradual oxidation of the carbon, hydrogen and nitrogen of the organic substance, by which these elements are converted into carbonic acid, water and nitric acid, respectively. In this process it is assumed that the carbon first combines with oxygen, after which the freed hydrogen and nitrogen unite to form ammonia, and any excess of hydrogen joins with oxygen to form water. The ammonia is next oxidized, its hydrogen passing into water, while its nitrogen passes first into nitrous acid and finally into nitric acid. These several acids in turn seize upon any mineral bases that may be present, and form carbonates, nitrites and nitrates. Of these various compounds, the ammonia and nitrites are intermediate products, and represent decomposition in progress. It may also be noted that in nature these chemical changes are the result of the activity of micro-organisms, and do not take place in their absence.

From the foregoing brief outline, we are enabled to understand why experts in water analysis attach so much importance to the amount and condition of the nitrogen which is found in the sample. Before decomposition begins, the nitrogen of organic matter is principally in the form of albumen or protoplasm, and has accordingly been designated by chemists as albuminoid ammonia ; but after decomposition has set in, the nitrogen is found successively as ammonia, nitrites and nitrates. Another important factor in the estimate of a water is the amount of chlorine which it contains, usually in the form of common salt, inasmuch as this is always associated with sewage or house drainage, and thus becomes in most cases the measure of the degree of pollution from such sources. It may also be remarked that the methods by which the amount of albuminoid ammonia is determined, afford no indication whether the source of the organic nitrogen is animal or vegetable matter ; and it is only by taking the various other factors into account that a sound inference in this respect can be reached. A relatively large quantity of albuminoid ammonia is, even when accompanied with much free ammonia, no evidence of sewage pollution, since in most surface waters it is derived from vegetable growths in the lakes, ponds and streams, and on their bottoms. On the other hand, when nitrites and chlorides are present in large quantity, it may be assumed that some dangerous pollution exists.

Various standards of purity, or rather allowable impurity, for potable water supplies have been proposed by eminent authorities, but no rigid rule has yet been generally adopted, owing to the uncertainties which still attend such work. In doubtful cases, accordingly, a valid verdict cannot be given until all possible collateral examinations, with respect to the surroundings, geology and natural history of the source, have been completed. For convenience of examination, however, a few of these standards have been arranged as compactly as possible in the appended Table A.

Prof. Wanklyn, one of the foremost English authorities on the subject, states that a water giving less than 0.005 part of albuminoid ammonia in 100,000 parts may be regarded as very pure ; while a water containing 0.005 part of such ammonia, together with a considerable quantity of free ammonia is suspicious, but in the absence of free ammonia, the albuminoid ammonia may be allowed to rise to nearly 0.010 part without causing the water to be regarded as bad. On the other hand, if this constituent is above 0.010 part in 100,000, the water should be regarded as very suspicious, and when it is over 0.015 part, the water must be condemned.

Dr. Frankland, another eminent English authority, makes the following general classification :

"Surface water or river water which contains in 100,000 parts more than 0.2 part of organic carbon, or 0.03 part of organic nitrogen, is not desirable for domestic supply, and ought, whenever practicable, to be rejected. Spring and deep-well water ought not to contain in 100,000 parts more than 0.10 part of organic carbon or 0.03 part of organic nitrogen. Impure water is such as exhibits more than 0.40 part of organic carbon."

In Tiemann & Gartner's exhaustive work on the "Examination of Water," which is regarded as the best German authority, the following requirements are made for a satisfactory quality of potable water :—

(a) The local conditions of the source should insure the permanence of an approximately uniform quality of the water and preclude the chance of pollution.

(b) The water should have no taste or odor, or rather its taste should be agreeable and cooling. It should also be clear and colorless when examined in a stratum of moderate depth. Its temperature should differ as little as possible from the mean annual temperature of the locality, in our latitudes, and should remain as uniform as possible throughout the year.

(c) Its chemical analysis should exhibit quantities not in excess of the following limits, expressed in parts per 100,000 :—A residue of 50 parts of combined mineral and organic matter, on evaporation at 212°F.; from 18 to 20 parts of alkaline earthy matter, such as oxides of lime and magnesium ; from 2 to 3 parts of chlorine, corresponding to 3.3 and 5 parts of chloride of sodium ; from 8 to 10 parts of sulphuric acid ; from 0.5 to 1.5 parts of nitric acid ; traces only of free ammonia and nitrous acid ; from 0.8 to 1.0 part of potassium permanganate to reduce the organic matter present ; 0.5 part of organic carbon ; 0.2 part of albuminoid ammonia ; hardness under 20°.

(d) Furthermore, the water should not contain in suspension considerable quantities of living or dead matter ; and if such suspended matter is traceable to human habitations, the water should be rejected. It should also be free from pathogenic bacteria ; and with regard to the harmless varieties of such organisms, a numerical measure may be derived from the consideration that the best natural spring waters usually contain no more than 50 bacteria per cubic centimeter, and that in the majority of unpolluted wells there are generally less than 500 bacteria in the same unit of volume ; while with properly conducted filtration through sand, the effluent water rarely has more than 100 bacteria per cubic centimeter.

Dr. Reuben Haines, of Philadelphia, recently offered the following figures representing the averages of 34 different determinations of uncontaminated waters, and recommended them as standards for pure waters in the neighborhood of that city :—

Free ammonia	0.0031	parts	in	100,000
Albuminoid ammonia	0.0044	"	"	"
Nitrogen as nitrates	0.5075	"	"	"
Chlorine	1.1900	"	"	"
Total solids	12.5700	"	"	"

Dr. Albert R. Leeds, of Hoboken, N. J., in reporting in 1882 upon the proposed water supply of Wilmington, offers the following criteria with respect to the waters of flowing streams, such as are used for most of the supplies of American cities :—

Free ammonia	0.001	to	0.012	parts	in	100,000
Albuminoid ammonia	0.010	"	0.028	"	"	"
Nitrogen as nitrates	0.111	"	0.389	"	"	"
Nitrogen as nitrites	0.0003	"		"	"	"
Chlorine	0.300	"	1.000	"	"	"
Total solids	15.00	"	20.00	"	"	"
Hardness	5.00	"	15.00	"	"	"

Concerning the modern bacteriological examination of water, there is now time to say but little, as this field is not only of great magnitude, but has also not yet been fully explored. After an experience of fifteen years in this direction, Prof. Miquel, perhaps the foremost of the French investigators, ventures to suggest the following classification of the water supplies of France, expressed in the number of bacteria per cubic centimeter :—

Extremely pure	0	to 10
Very pure	10	to 100
Pure	100	to 1000
Medium	1000	to 10,000
Impure	10,000	to 100,000
Very impure, above		100,000

Enough has doubtless been said in the foregoing to indicate that the determination of a suitable source of supply is by no means a simple matter, but requires a vast amount of careful study, so far as quality alone is concerned ; and as there are numerous other matters of interest which remain to be considered, we will next proceed to the subject of the proper quantity of water that should be provided each day for each head of the population.

Many attempts have been made to estimate this quantity from purely theoretical considerations, on the basis of allowing a certain number of gallons per day for each individual of the community for domestic purposes, and adding thereto the amount assumed to be needed for manufacturing, commercial and general public uses ; also by taking into account the probable future growth of population and industries. Few of such estimates have, however, been found to be correct, and the best that can be done is to be guided by the experience gained in other places. To show the wide diversity which exists in the per capita water consumption of different cities of various magnitude, the appended Table B has been compiled ; and it may be added that the figures therein given are derived from the best available authorities, and may be regarded as the most accurate which have yet been obtained.

This table exhibits the population, and average daily water consumption per capita, in 276 American and foreign cities, with whose names at least we are all familiar. The aggregate population here represented is nearly 46,700,000, of which about 16,600,000 is American, and 30,100,000 foreign. This vast number of people consumes on the average 2,934,000,000 gallons of water every day, thus giving to each person a daily average of 62.8 gallons. The list embraces 100 American and 176 foreign cities, the former using about 1,863,000,000 gallons per day, with an average of 112 gallons per head, and the latter consuming about 1,071,000,000 gallons per day, with an average of 35.6

gallons per head ; hence it is seen that the average daily per capita consumption in American cities is more than three times greater than in foreign towns. No good reason for this great difference in consumption can be given, except that in many foreign cities a large proportion of the water which is used for manufacturing purposes is taken by the owners of such establishments from private wells or streams so that it does not appear in the report of the public supply, and that the quantity wasted by the inhabitants is much less than in our communities.

In the list we also find four great American cities with an average population of 1,475,000, taking an average daily per capita supply of 106 gallons ; and four great European cities with an average population of 2,146,000, taking an average of 39.4 gallons per capita. The three to one ratio is here again apparent. Furthermore, the remaining 96 American cities, with an aggregate of 10,682,000 people, consume on the average 115.9 gallons per capita ; while the remaining 172 foreign cities, with a total of 21,525,000 inhabitatants, use only 34.1 gallons per capita per day on the average ; the ratio in this case being 3.4 to 1. Many other interesting comparisons might be made from this exhibit, but they must be omitted for want of time.

The principal points to be observed in determining the proper quantity of water which should be supplied to any particular community, are : (1) The amount needed for purely domestic purposes ; (2) the quantity required for the special industries that may have been developed therein, and which it is desirable to retain ; (3) the amount needed for the supply of public buildings, such as schools, hospitals, baths and other similar institutions, together with that which is required for other public purposes, such as watering streets, the suppression of fires, and for parks, ornamental fountains, etc. ; (4) the loss by unavoidable leakage in conveyance and distribution, along with a margin for justifiable waste in different seasons ; and lastly (5) the allowance for the future growth of the city and its industries, coupled with the prospective deterioration or fouling of the conduit and the consequent reduction of its discharging capacity.

A full discussion of each of these items is impracticable at the present time, and hence we will consider only the last mentioned ones as briefly as possible. In the case of a comparatively old and well established city of over 50,000 inhabitants, in which a system of water works is to be introduced, it is fair to presume that after the successful completion of the works, the increase of population will proceed at a somewhat higher rate than previously, inasmuch as many industrial enterprises will be attracted to, or be developed in the community, in consequence of the superior advantages afforded by the facility of obtaining water for business purposes and protection against fire. For the same reason also, the land-owners are stimulated to erect a better class of buildings than formerly, thereby making the city handsomer and inducing others to settle therein to enjoy the conveniences thus offered. The impulse so acquired is usually retained for a long term of years, and may, under favorable conditions of the country, even be considerably accelerated. In estimating the probable rate of growth of such a city, after the introduction of an ample and wholesome water supply, we are accordingly justified in adopting a figure which may seem alarmingly high to conservative minds.

On the other hand, when we are concerned with the enlargement of an existing supply, it may safely be assumed that the previously established rate of growth will remain at least constant for a considerable number of years. In this case, therefore, the future population at any time during such period may be computed by the rule for ordinary compound interest, and it may also be assumed that the per capita consumption of water will increase somewhat, owing to the development of unforeseen industries, or sources of uncontrollable waste. The required additional volume of water must accordingly be estimated with much care in the light of the past experience with the particular works, also bearing in mind the results found in other cities of similar size under like circumstances.

Where the supply of a city is obtained by gravity from a distant point, as in the case of Rochester, N. Y., it may become expedient to make the additional quantity to be taken dependent upon a variety of other conditions besides those heretofore mentioned. The capacity of the source and the cost of the new works, must here be taken into account, as well as the further serviceability of the old conduit. Here there was some doubt about the continued usefulness of a considerable portion of the old pipe, and hence the first tendency was to advocate the construction of a new line of sufficient capacity to convey to the city the entire yield of the watershed of the two mountain lakes from which the supply is taken. The estimated cost of such a conduit, however, was much larger than had been anticipated by the municipal authorities and interested citizens, and the question then became essentially one of financial expediency in view of the existing bonded indebtedness. New computations of the average yield of the entire available drainage area were then made, and it was finally decided that it would be better to make the ultimate capacity of the contemplated pipe line equal to one-half of such average yield, inasmuch as this quantity would probably be adequate for at least 20 years, after which time the remaining half could be taken in a duplicate pipe to be placed alongside of the new one.

By this means there would finally be two similar lines of pipe from the lake to the city, it being also presumed that the upper section of the original conduit would become practically useless or worn out by the time that the second additional supply might be required ; furthermore, with two such lines, the dangers and grave inconveniences of a complete interruption of the supply, in case of necessary repairs to a single conduit, would be almost wholly averted, as it rarely happens that both pipes are out of order simultaneously ; and finally, the cost of the undertaking would fall within the expedient limit of expenditure, while the quantity of water thus procured would be eminently satisfactory, especially if the old conduit could be retained in active service.

The computations for the average yield of the entire drainage area showed that in a cycle of three driest years, a uniform supply of about 32,000,000 gallons per day might be depended on, the storage capacity of the two lake basins being taken into account. The pipe portion of the new conduit was therefore given an ultimate capacity of 16,000,000 gallons per day, and it was assumed that during the next 20 years, the average daily per capita consumption of water in the city from the gravity system would increase from 60 to 80 gallons. It was also assumed that the capacity of the old conduit would become reduced to about 6,000,000 gallons per day, thus making the total available gravity supply at the end of the period 22,000,000 gallons per day, which at the rate of 80 gallons per head, would suffice for a population of 275,000. In 1892, however, the census returns showed that the city had about 145,000 inhabitants, and that the increase had then proceeded at the rate of about 5,000 persons, or 3.4 per cent. per year. Applying the rule for ordinary compound interest, it is found that at this rate of growth the population of the city at the end of 20 years should be about 283,000, and that at the present time, 5½ years after the said enumeration was taken, it should be about 174,300.

It will thus be seen that in 1892, when the new conduit was designed and its construction authorized, provision was made for an increase of nearly double the then-existing population, together with a large increase in the average daily per capita consumption ; and it now remains to be determined whether these assumptions will be verified. No good proof is yet available, as no census has been taken since 1892 ; but from the usual ratio of population to votes cast at important State or National elections, it is found that the above computation of the present number of inhabitants is substantially correct ; and it is also known that the average per capita use of water is now about 70 gallons per day. It may also be mentioned that by extending the application of water meters, a great proportion of preventable waste from the water system can be stopped, thus prolonging considerably the period of time before another supply will be needed.

Having dealt briefly with the preliminaries relating to the quality and quantity of our water, we may next refer to the general design and construction of the works. As is well known, the present source of the domestic supply is Hemlock Lake, a beautiful body of clear and wholesome water situated in a deep valley of a mountainous or hilly district about 29 miles south of the city. In form, the lake is roughly a long rectangle, 6.6 miles in length and 0.6 mile wide, with an area of about 1,828 acres at ordinary low water, and an average depth of 65 feet along its middle line. It lies at an elevation of about 386 feet above the general level of the city, and including the water surface, its drainage area is 43.05 square miles in extent. Adjacent thereto on the east, and 200 feet higher, is Canadice Lake, a similar sheet of water with an area of 648 acres and a watershed of 13 88 square miles. The outlet of this lake empties into that of Hemlock Lake near the foot of the latter, and can easily be diverted directly therein, so that its drainage area of 5.49 square miles may also be made available as a gathering ground for the city's supply. The total area thus capable of use is nearly 40,000 acres, or 62.43 square miles; but up to the present time, no draft upon the Canadice watershed has been necessary.

The old conduit from the lake to the city was built during the years 1873, '74 and '75. Excluding the intake pipe, which extends 1,000 feet out from the shore into moderately deep water, it consists of 9.63 miles of 36-inch wrought-iron riveted pipe, made of plates $\frac{3}{16}$-inch thick; 2.92 miles of 24-inch riveted pipe, made of plates $\frac{3}{16}$-inch and $\frac{1}{4}$-inch thick; and 15.36 miles of 24-inch cast-iron pipe of varying thickness according to the pressure. These lengths are measured from the gate house at the lake to that at Mt. Hope reservoir, and if we include the inlet and outlet pipes at the lake, and the two reservoirs, the total length of pipe is 28.25 miles. It was deemed expedient to divide this long distance into at least two parts by an intermediate reservoir, and advantage was accordingly taken of the line of hills which extends through the townships of Rush and Henrietta, about 9 miles south of the Mt. Hope ridge, to build the storage basin known as Rush reservoir. The capacity of this reservoir is about 63,500,000 gallons, while the distributing reservoir at Mt. Hope contains about 22,500,000 gallons. The elevations of these two bodies of water above the general level of the city are respectively 240.3 feet and 124.4 feet.

The new conduit from the lake to the city was constructed during the years 1893 and '94. It consists of the following principal parts: (1) A riveted steel plate intake pipe, 5.0 feet in diameter and 1,550 feet long, extending from the shore of the lake out into deep water ; (2) A brick conduit of horse-shoe shaped cross-section, 6 feet high and wide inside, and 2.25 miles long, of which about 1.45 miles was built in tunnel through tough clay and shale rock ; (3) A continuous riveted steel pipe 38 inches in diameter and 17.36 miles long to Rush reservoir ; (4) A line of 36-inch cast-iron pipe about 0.20 mile long ; and lastly, (5) A continuous riveted steel pipe, 38 inches in diameter and 8.55 miles long terminating at Mt. Hope reservoir. The plates of the steel pipe are $\frac{1}{4}$, $\frac{5}{16}$ and $\frac{3}{8}$-inch thick, according to their location with respect to pressure.

A few words may also be used to explain why the new intake pipe and the brick conduit were made of such relatively large size. When the old conduit was laid through the alluvial lands extending northerly from the foot of the lake, enormous difficulties with quicksand were encountered. From the borings made preliminary to the construction of the new works, it was found that strata of this treacherous material were widely distributed in the subsoil of the valley, and that the cost of building a suitable masonry conduit along the lowest available routes would be so great, that the addition of 1$\frac{1}{2}$ or 2 feet to the diameter needed for a delivering capacity of 16,000,000 gallons per day, would not involve a serious increase of expense ; and as this additional size would give the conduit a capacity for discharging easily 32,000,000 gallons per day, which was the computed average yield of the entire watershed of the aforesaid two lakes, it was deemed prudent to adopt the larger dimensions in order that no further outlay in this locality would arise if a second additional water supply should ever be required in the future. The same considerations apply with even greater force to the case of tunnel work, owing to the absolute necessity of securing room enough for the workmen to perform their labor properly and quickly. It may also be mentioned that carefully prepared estimates of cost by various routes, showed that the expense of building the conduit by tunneling through the clay and firm rock in the side of the valley, would be no greater, and might easily be less, than that of construction through the earth and quicksand of the bottom lands.

[NOTE.—A few data of general interest relating to the distribution of the water within the city limits were then given, after which a large number of photographic views taken during the construction of the new conduit were exhibited and explained. Some of these views, which represent the characteristics of the different sections of the work, are appended.]

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TABLE A.—APPROXIMATE STANDARDS FOR ESTIMATING THE QUALITY OF A WATER SUPPLY.

Chemical Constituents.		Proposed by Various English Authorities. (Parkes' Hygiene, etc.)				Proposed by Dr. Leeds.	Proposed by Dr. Haines.	Proposed by Tiemann and Gärtner.
		Parts per 100,000 in						
		1. Pure and Wholesome Water.	2. Usable Water.	3. Suspicious Water.	4. Impure Water.	5. Wholesome Waters of American Rivers.	6. Wholesome Waters in Vicinity of Philadelphia.	7. Usable Water.
1. Chlorine in chlorides	under	1.4000	4.2857	4.0 to 7.0	More than 7.1428	0.3000 to 1.000	1.1900	2.0 to 3.0
2. Solids in solution: total	"	7.1428	42.8571	43. to 71.	71.4285	15.00 to 20.00	12.5700	50.0
Solids in solution: volatile	"	1.4000	4.2857	4. to 7.	7.1428			30.0 to 32.0
3. Ammonia, free or saline	"	0.0020	0.0050	0.0050 0.0100	0.0100	0.0010 to 0.0120	0.0031	Trace
Ammonia, Albuminoid	"	0.0050	0.0100	0.0100 0.0125	0.0125	0.0100 to 0.0280	0.0044	0.20
4. Nitric acid, in nitrates	"	0.0323	0.5000	0.5 to 1.0	1.0000			0.5 to 1.5
Nitrous acid, in nitrites	"	None	None	0.0500	0.0500	0.0002		Trace
Nitrogen in nitrates	"	0.0140	0.1129	0.1243 to 0.2373	0.2415	0.111 to 0.389	0.5075	
Total combined nitrogen, incl. ammonia free	"	0.0160	0.1170	0.1247 to 0.2455	0.2497			
Total combined nitrogen, including that in the albuminoid ammonia	"	0.0230	0.1252	0.1255 to 0.2465	0.2601			
5. Oxygen absorbed by organic matter within 30 minutes at 140° F.	"	0.0250	0.1000	0.10 to 0.15	0.1500			
Same, in 15 minutes at 80° F.	"	0.0125	0.0300	0.05 to 0.10	0.1000			
Same, in 4 hours at 80° F.	"	0.0500	0.1500	0.2 to 0.4	0.4000			
6. Hardness, total	"	8.5°	17.3°	Over 17°	28.5°	5° to 15°		20°
Hardness, fixed	"	3.0°	5.7°	Over 5.7°	8.7°			
7. Phosphoric acid, in phosphates	"	Trace	Trace	Heavy traces	Very heavy traces.			
Sulphuric acid, in sulphates	"	Trace	3.0000	3.0000	4.2857			8.0 to 10.0
8. Heavy metals	"	None	Trace	Traces	Any except iron.			
9. Hydrogen sulphide in alkaline sulphides		None	None	None	Present			
10. Bacteria, per cubic centimeter (Miquel)		100 to 1,000	1,000 to 10,000	10,000 to 100,000	Over 100,000			50 to 500

NSUMPTION

—FROM A COMPILA LY,"

Name of Cit	rage ump- per ad Gall's.
Berlin	7
Hamburg	7
Breslau	9
Schweidnitz	2
Dresden	0
Leipzig	3
Cologne	9
Königsberg	3
Hannover	0
Altona	3
Bremen	9
Magdeburg	6
Nürnberg	6
Düsseldorf	0
Chemnitz	6
Dortmund	4
Stettin	6
Danzig	5
Aachen	4
Strassburg	2
Braunschweig	
Crefeld	9
Elberfeld	2
Liegnitz	
Halle	
Essen	
Posen	
Mühlhausen	
Augsburg	
Cassel	
Metz	
Lübeck	
Frankfort a / O	
Würzburg	
Gõrlitz	
Wiesbaden	
Carlsruhe	rage
Potsdam	ump-
Bochum	per
Mühlheim and De	ad
Kiel	Gall's.
Gelsenkirchen	
Bonn	.9
Duisburg	.7
Darmstadt	3.5
Münster	5.5
Gladbach	.9
Rostock	2.8
Freiberg	7.3
Regensburg	4.0
Flensburg	2.0
Charlottenburg	3.5
Bamberg	5.1
Nordhausen	1.4
Heilbronn	
Dessau	7.6

6. OTHER COUNTRIES.

FROM BAZALGETTE'S PRESIDENTIAL ADDRESS BEFORE ENGLISH INSTITUTE OF CIVIL ENGINEERS, IN JAN., 1884.

No.	Name of City.	Estimated Population 1883.	Average Consumption per head U. S. Gall's.
	I. EUROPEAN CITIES.		
1.	Madrid, Spain	397,800	4.0
2.	Barcelona, Spain	370,600	17.6
3.	Seville, Spain	132,800	8.7
4.	Lisbon, Portugal	187,400	18.2
5.	Antwerp, Belgium	190,000	20.9
6.	Liege, Belgium	129,200	19.8
7.	Amsterdam, Holland	350,200	13.4
8.	Rotterdam, Holland	162,100	26.9
9.	The Hague, Holland	130,000	21.1
10.	Copenhagen, Denmark	234,900	16.7
11.	Christiana, Norway	122,100	47.6
12.	Stockholm, Sweden	176,700	16.2
13.	St. Petersburg, Russia	929,100	25.6
14.	Riga, Russia	169,300	37.3
15.	Vienna, Austria	740,600	23.8
16.	Buda-Pesth, Austria	395,900	39.6
17.	Munich, Bavaria	240,000	39.6
18.	Frankfort a / M, Germany	141,000	28.8
19.	Zurich, Switzerland	96,700	56.7
20.	Basel, Switzerland	74,500	34.8
21.	Lausanne, Switzerland	23,300	63.4
22.	Athens, Greece	90,000	25.9
23.	Rouen, France	105,900	37.0
	Total and average for 23 cities	5,590,300	23.8
	II. OTHER FOREIGN CITIES.		
24.	Cairo, Egypt	450,000	13.2
25.	Alexandria, Egypt	222,800	22.8
26.	Bombay, India	773,200	24.0
27.	Calcutta, India	433,200	26.4
28.	Delhi, India	173,400	19.2
29.	Lahore, India	128,000	12.0
30.	Kurrachee, India	68,300	30.0
31.	Adelaide, Australia	40,000	60.0
32.	Brisbane, Australia	30,000	40.0
33.	Rio de Janeiro, Brazil	350,000	36.0
34.	Pernambuco, Brazil	120,000	8.8
35.	Valparaiso, Chili	96,000	24.0
	Total and avg. for 12 other cities	2,884,900	23.4
	Total & avg. for 35 foreign cities	8,475,200	23.7

7. GENERAL SUMMARY.

No. of Cities.	Countries.	Estimated Population.	Average Consumption per head in U. S. Gallons.
100	American and Canadian	16,582,100	112.0
176	European and foreign	30,108,800	35.6
276	Total considered	46,690,900	62.8

Total water consumed per day, 2,934,098,800 gallons.

TABLE B. SHOWING AVERAGE DAILY WATER CONSUMPTION PER HEAD OF POPULATION IN 270 AMERICAN AND FOREIGN CITIES

1. UNITED STATES AND CANADA

[illegible]

No.	Name of City	Estimated Population	Average Consumption per head U. S. Galls.
[illegible]	McKeesport, Pa.	[illegible]	[illegible]
	Total and average	[illegible]	12[illegible]

Total water consumed per day [illegible] gallons

2. GREAT BRITAIN

From M. Hyde's [illegible] Statistics of British Water Works [illegible]

No.	Name of City	Estimated Population	Average Consumption per head U. S. Galls.
	I. ENGLAND		
[illegible]	Scarborough	[illegible]	[illegible]
[illegible]	Tunbridge Wells	[illegible]	[illegible]
[illegible]	Whitehaven	[illegible]	[illegible]
[illegible]	Gainsborough	[illegible]	[illegible]
	Total and average	[illegible]	[illegible]
	Exclusive London	[illegible]	[illegible]
	II. SCOTLAND		
[illegible]	Glasgow	[illegible]	[illegible]
[illegible]	Edinburgh	[illegible]	[illegible]
[illegible]	Aberdeen	[illegible]	[illegible]
[illegible]	Perth	[illegible]	[illegible]
4[illegible]	Hamilton	[illegible]	48.[illegible]
41	Ayr	[illegible]	84.0
42	Inverness	[illegible]	[illegible]
	Total and average	[illegible]	[illegible]
	III. IRELAND		
43	Belfast	255,[illegible]	42.[illegible]
	Total and average for 43 British cities	9,616,400	3[illegible].3

3. GERMANY

I. From [illegible] compiled by A. Thiem in Journal für Gasbeleuchtung und Wasserversorgung No. [illegible]

No.	Name of City	Estimated Population	Average Consumption per head U. S. Galls.
[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	Leipzig	[illegible]	[illegible]
[illegible]	Hannover	[illegible]	[illegible]
[illegible]	Magdeburg	[illegible]	[illegible]
[illegible]	Nürnberg	[illegible]	[illegible]
[illegible]	Düsseldorf	[illegible]	[illegible]
[illegible]	Chemnitz	[illegible]	[illegible]
[illegible]	Dortmund	[illegible]	[illegible]
21	Braunschweig	[illegible]	[illegible]
[illegible]	Crefeld	[illegible]	[illegible]
[illegible]	Elberfeld	[illegible]	[illegible]
24	Liegnitz	[illegible]	1[illegible].3
25	Halle	[illegible]	[illegible]
26	Essen	[illegible]	[illegible]
27	[illegible]	[illegible]	[illegible]
28	Mühlhausen	[illegible]	[illegible]
29	Augsburg	[illegible]	[illegible]
[illegible]	Cassel	[illegible]	[illegible]
[illegible]	Metz	53,[illegible]	[illegible]
[illegible]	Lübeck	[illegible]	[illegible]
[illegible]	Frankfurt a. O.	[illegible]	[illegible]
34	Würzburg	[illegible]	[illegible]
[illegible]	Görlitz	[illegible]	[illegible]
[illegible]	Wiesbaden	[illegible]	17
[illegible]	Carlsruhe	[illegible]	[illegible]
[illegible]	Potsdam	[illegible]	[illegible]
[illegible]	Bochum	[illegible]	[illegible]
40	Mühlheim and Deutz	[illegible]	13.2
41	Kiel	[illegible]	10.6
42	Gelsenkirchen	43,000	[illegible]
43	Bonn	[illegible]	[illegible]
44	Duisburg	41,2[illegible]	28.[illegible]
45	Darmstadt	[illegible]	[illegible]
46	Münster	[illegible]	[illegible]
47	Gladbach	37,[illegible]	[illegible]
48	Rostock	[illegible]	[illegible]
49	Freiburg	[illegible]	[illegible]
50	Regensburg	34,500	38.0
51	Flensburg	[illegible]	10.5
52	Charlottenburg	[illegible]500	12.9
53	Bamberg	29,[illegible]	[illegible]
54	Nordhausen	26,200	9.1
55	Heilbronn	24,[illegible]	19.8
56	Dessau	2[illegible],300	1[illegible].2
[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	Neisse	[illegible]	[illegible]
[illegible]	Ratibor	[illegible]	[illegible]
[illegible]	Neuss	[illegible]	[illegible]
[illegible]	Oberhausen	[illegible]	[illegible]
[illegible]	Iserlohn	[illegible]	2
	Total and average	[illegible]	[illegible]
	For Nos. [illegible]	[illegible]	[illegible]

II. From [illegible]'s Treatise on Water Supply, 1896

No.	Name of City	Estimated Population 1890	Average Consumption per head U. S. Galls.
1	Berlin	1,4[illegible]	18.[illegible]
2	Breslau	[illegible]	2[illegible].1
3	Cologne	2[illegible]1,700	[illegible]4.3
4	Dresden	27[illegible],500	2[illegible].4
5	Nürnberg	145,000	[illegible]
6	Düsseldorf	144,600	25.5
7	Stuttgart	1[illegible],800	26.0
8	Elberfeld	135,000	26.0
9	Halle a. S.	1[illegible],400	23.[illegible]
10	Braunschweig	1[illegible],000	18.[illegible]
11	Dortmund	[illegible],700	[illegible]8.5
12	Lübeck	6[illegible],600	52.8
13	Wiesbaden	62,000	19.6
14	Duisburg	55,[illegible]	46.0
	Total and average	3,352,400	23.8
	Exclusive Berlin	1,925,200	28.1

4. FRANCE

From Bechmann's Treatise on Water Supply, Paris 1888

No.	Name of City	Estimated Population 188[illegible]	Average Consumption per head U. S. Galls.
1	Paris	2,2[illegible]	[illegible]
2	Lyon	31[illegible]	37
3	Marseille	[illegible]	[illegible]
4	Toulouse	1[illegible],[illegible]42	[illegible]2
5	Nantes	22,2[illegible]	[illegible]
6	Limoges	[illegible]	[illegible]
7	Besançon	51,[illegible]4	[illegible]9
8	Orléans	52,157	5[illegible]
9	Tours	[illegible]8,[illegible]25	5[illegible]
10	Dijon	[illegible]	[illegible]3
[illegible]	[illegible]	41,27[illegible]	2[illegible]
[illegible]	Boulogne	[illegible]	[illegible]
[illegible]	Laval	[illegible]	[illegible]
14	Arras	26,764	3[illegible]
15	Carcassonne	25,[illegible]	[illegible]5
16	Périgueux	[illegible]	34
17	Agen	[illegible]	[illegible]
18	Melun	[illegible]2[illegible]	[illegible]
19	Aurillac	[illegible]	4
20	Semur	4,[illegible]3	42
	Total and average	3,6[illegible]2,280	5[illegible]
	Exclusive Paris	1,40[illegible],263	62

* Denotes inadequate supply

5. ITALY

From Spataro's Treatise on "Hygiene of Habitations," 1892

No.	Name of City	Estimated Population 1891	Average Consumption per head U. S. Gall's
1	Naples	530,9[illegible]	90.9
2	Rome	423,2[illegible]	17[illegible].7
3	Turin	320,8[illegible]	18.5
4	Palermo	257,[illegible]	25.5
5	Genoa	206,500	90.9
6	Florence	191,500	22.8
7	Venice	1[illegible]8,[illegible]	17.3
8	Bologna	1[illegible],500	11.0
9	Catania	118,5[illegible]	12.[illegible]
10	Ferrara	[illegible]4,5[illegible]	18.5
11	Padua	7[illegible],5[illegible]	76.1
12	Spezia	24,2[illegible]	4.4
	Total and average	2,533,600	6[illegible].6

6. OTHER COUNTRIES

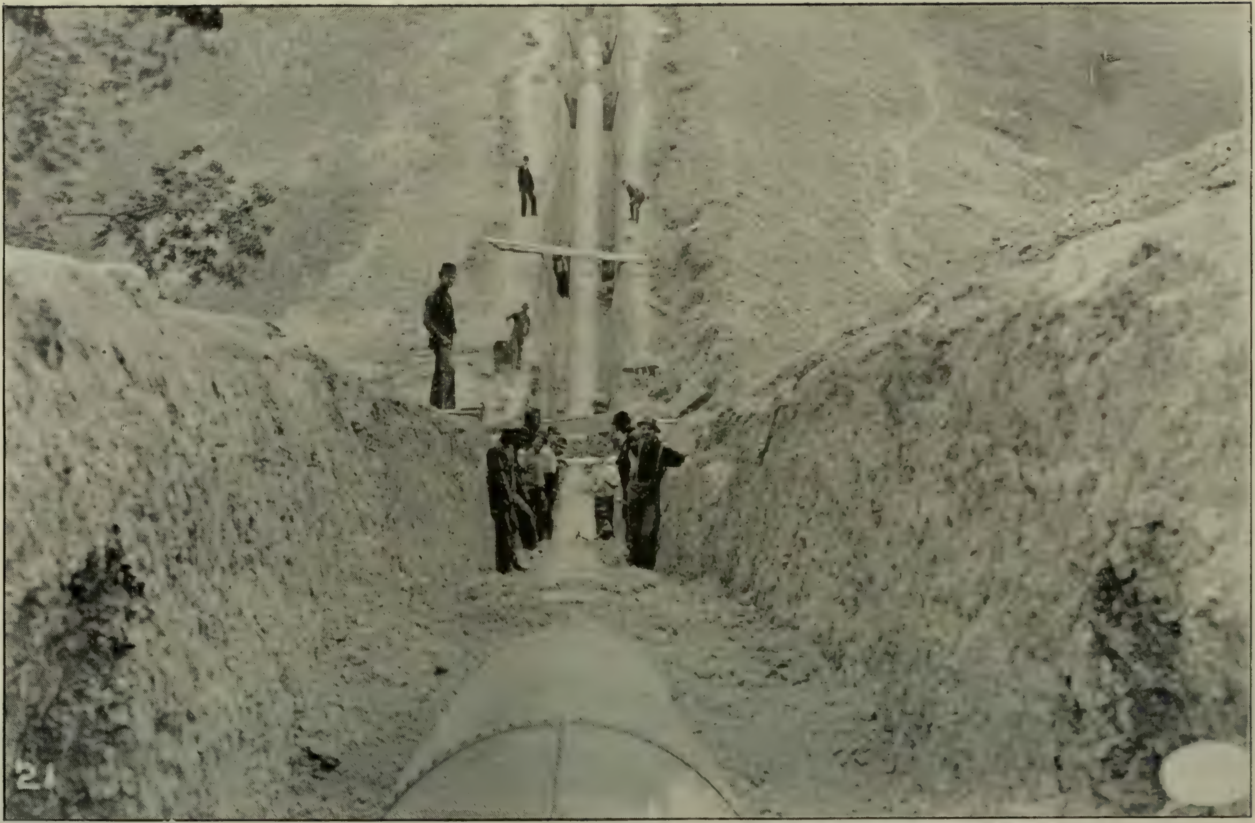
From Bazalgette's Presidential Address to English Institute of Civil Engineers in Jan. 1884

No.	Name of City	Estimated Population 188[illegible]	Average Consumption per head U. S. Galls.
	I. EUROPEAN CITIES		
1	Madrid, Spain	39[illegible],800	4[illegible]
2	Barcelona, Spain	[illegible]	1[illegible].6
3	Seville, Spain	1[illegible]2,800	[illegible]
4	Lisbon, Portugal	18[illegible]	18.2
5	Antwerp, Belgium	[illegible]	20.[illegible]
6	Liege, Belgium	1[illegible]0,200	19.8
7	Amsterdam, Holland	3[illegible],200	[illegible]4
8	Rotterdam, Holland	16[illegible]	[illegible]
9	The Hague, Holland	[illegible]	[illegible]
10	Copenhagen, Denmark	2[illegible]4,[illegible]	[illegible]
11	Christiania, Norway	122,[illegible]	17.6
12	Stockholm, Sweden	[illegible]	16.2
13	St. Petersburg, Russia	[illegible]	25.[illegible]
14	Riga, Russia	[illegible]	[illegible]7.3
15	Vienna, Austria	[illegible]	2[illegible].8
16	Buda Pesth, Austria	[illegible]	36.6
17	Munich, Bavaria	24[illegible]	[illegible].6
18	Frankfort a. M., Germany	1[illegible],000	28.8
19	Zurich, Switzerland	[illegible]	56.7
20	Basel, Switzerland	[illegible]	[illegible]4.8
21	Lausanne, Switzerland	23,[illegible]	6[illegible].4
22	Athens, Greece	[illegible]	25.9
23	Rouen, France	[illegible]	37
	Total and average for 23 cities	[illegible]	23.8
	II. OTHER FOREIGN CITIES		
24	Cairo, Egypt	45[illegible]	13.2
25	Alexandria, Egypt	222,800	22.8
26	Bombay, India	7[illegible],200	24
27	Calcutta, India	43[illegible]	26.4
28	Delhi, India	17[illegible]	19.2
29	Lahore, India	128,[illegible]	12
30	Kurrachee, India	6[illegible]	30[illegible]
31	Adelaide, Australia	[illegible],000	60.0
32	Brisbane, Australia	[illegible]	40.[illegible]
33	Rio de Janeiro, Brazil	350,[illegible]	36.0
34	Pernambuco, Brazil	12[illegible]	8.8
35	Valparaiso, Chili	[illegible]	24.0
	Total and avg. for 12 other cities	2,8[illegible]4,9[illegible]	25.4
	Total & avg. for 35 foreign cities	8,47[illegible],200	23.7

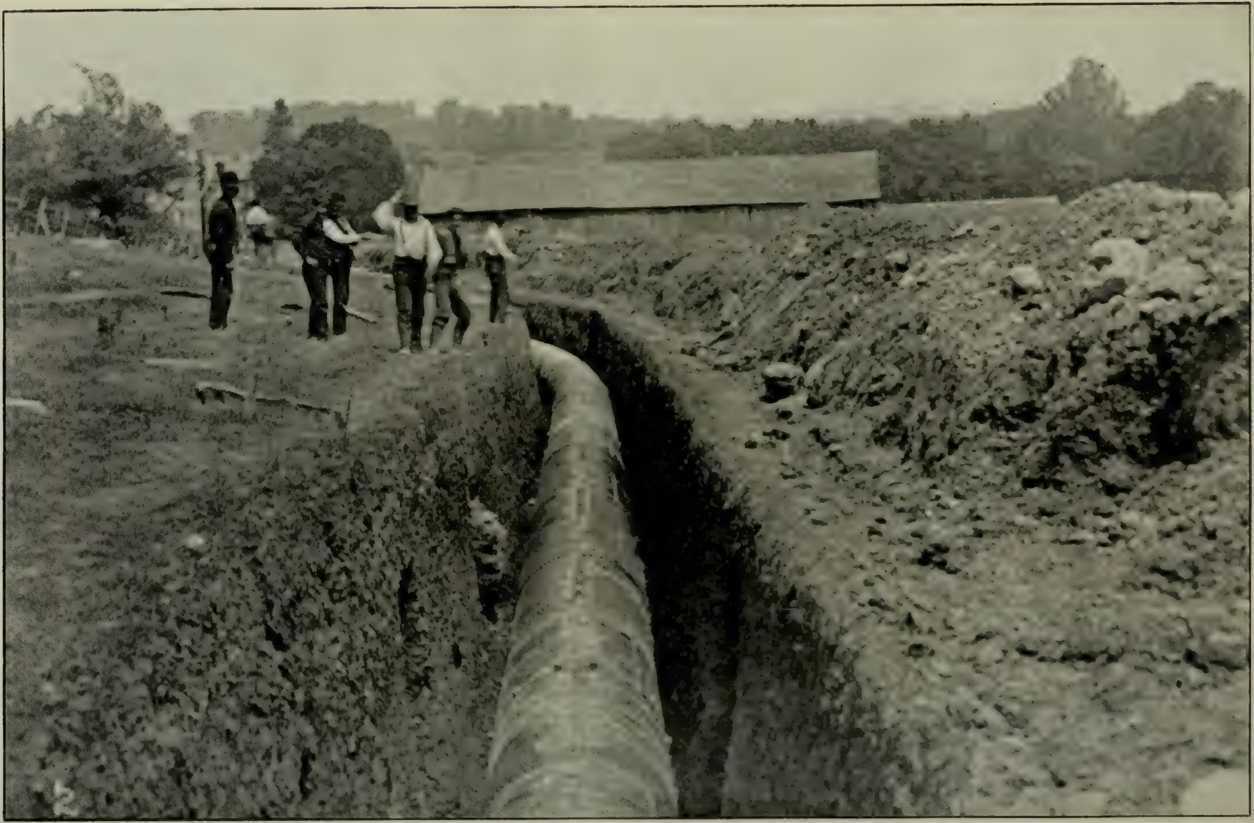
7. GENERAL SUMMARY

No. of Cities	Countries	Estimated Population	Average Consumption per head in U. S. Gallons
100	American and Canadian	16,5[illegible]2,100	112.0
176	European and foreign	30,108,800	35.6
276	Total considered	46,660,900	62.8

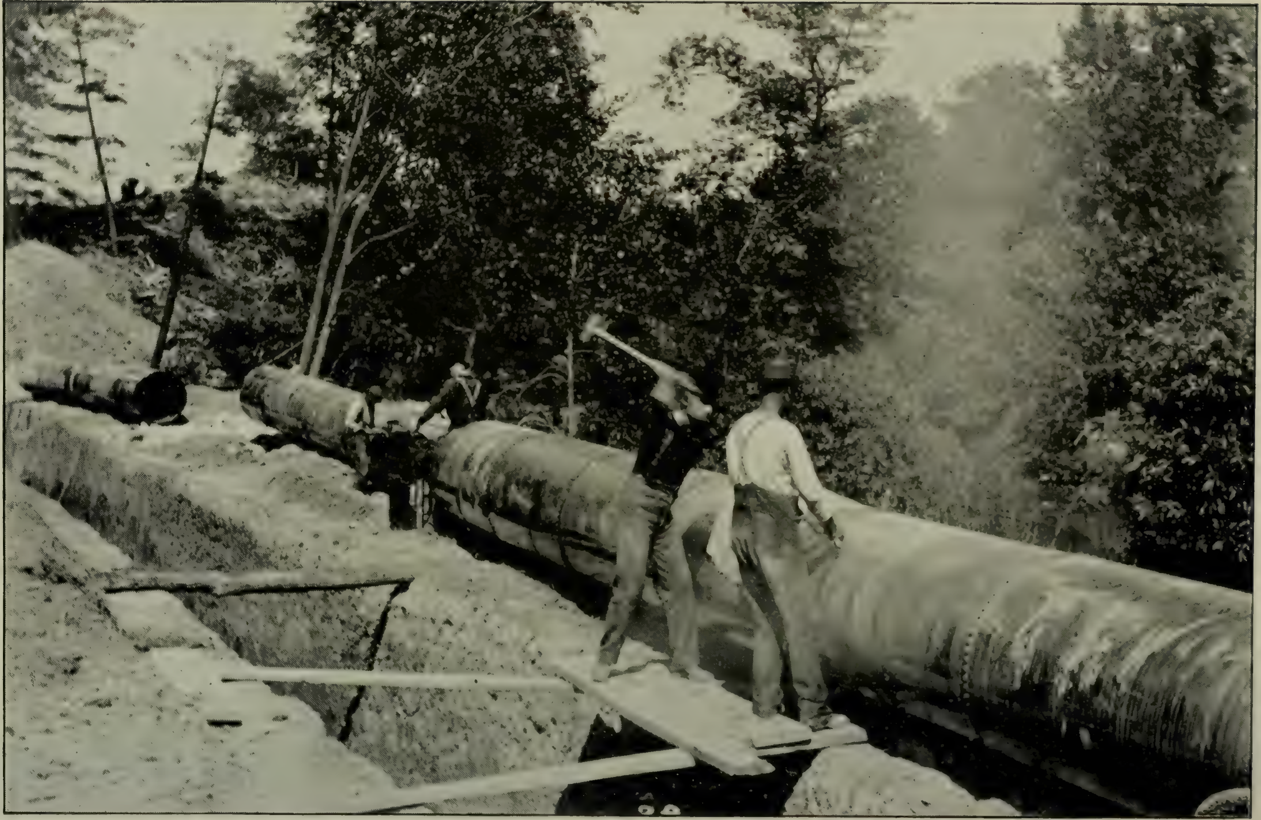
Total water consumed per day, 2,931,098,800 gallons



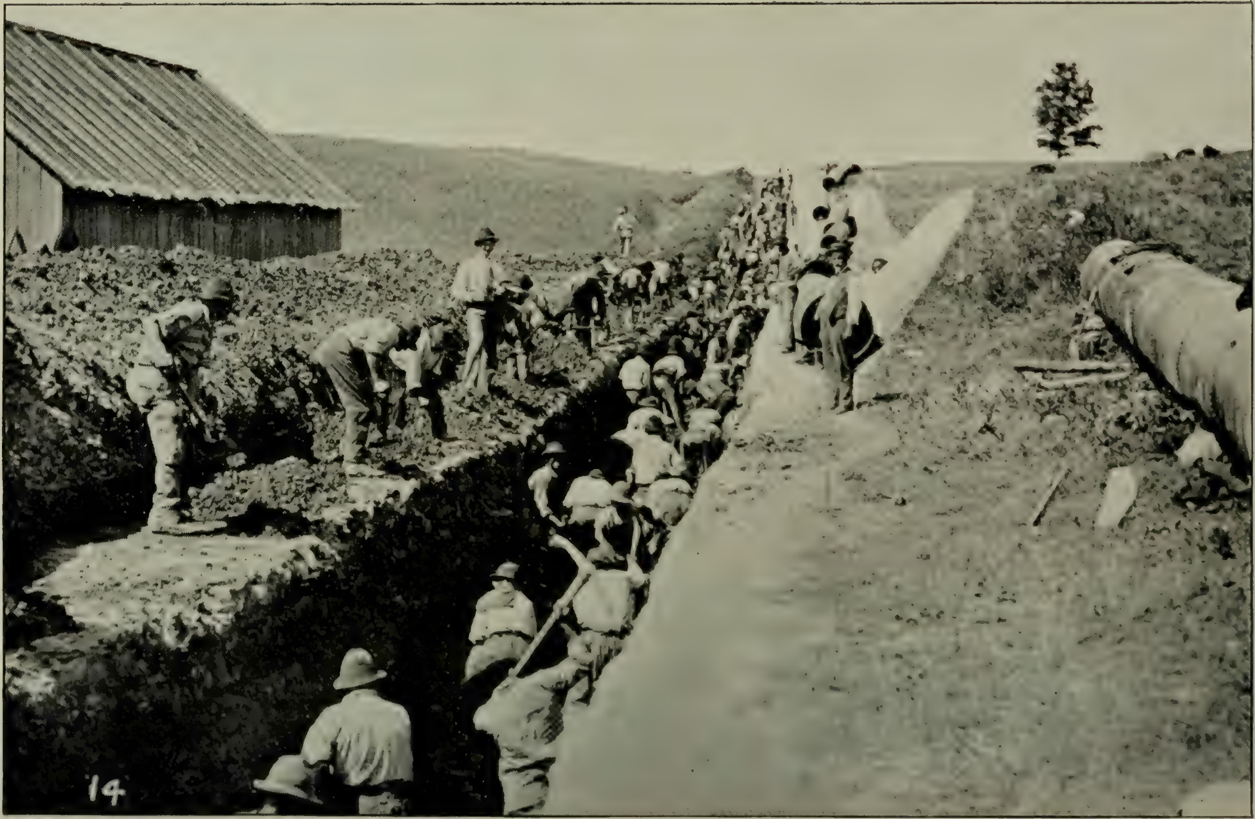
8.—VIEW SHOWING A VERTICAL CURVE IN THE 38-INCH STEEL PIPE, AS LAID.



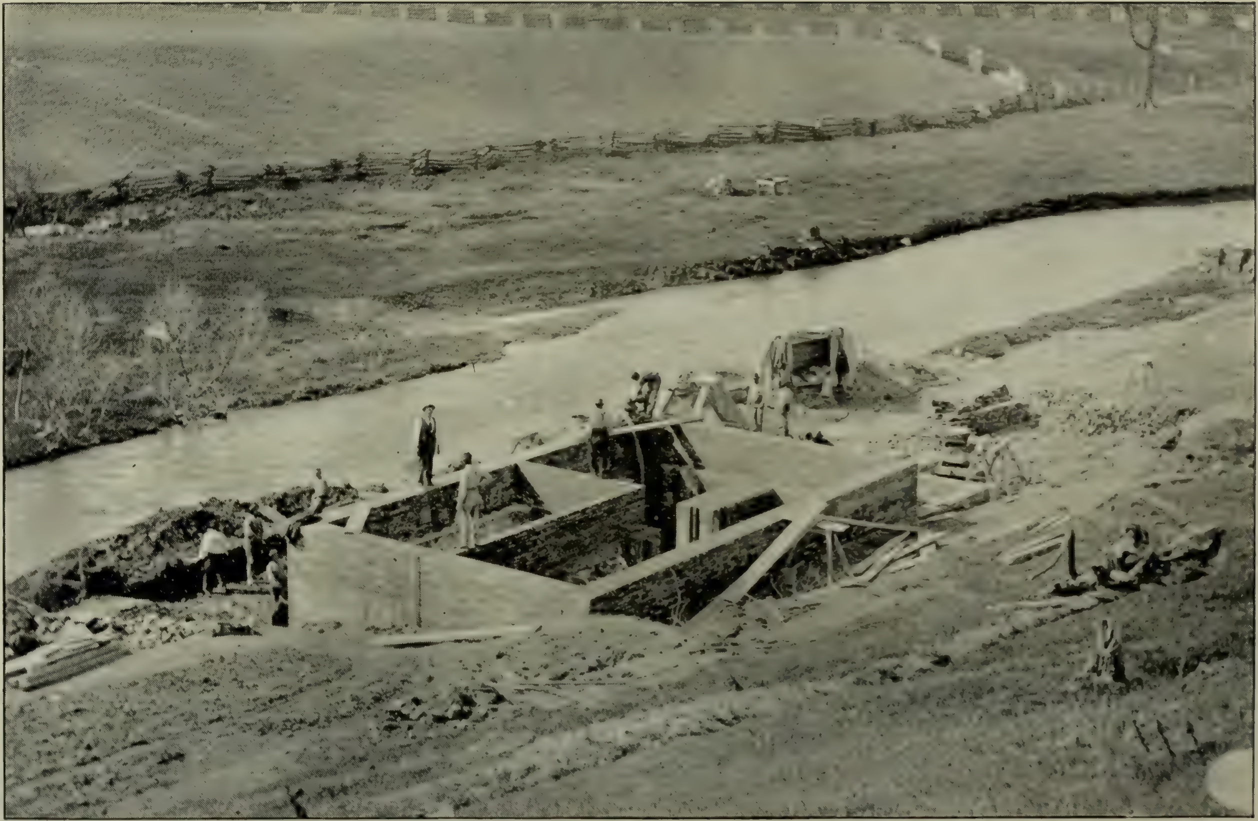
7.—VIEW SHOWING A HORIZONTAL CURVE IN THE 38-INCH STEEL PIPE, AS LAID.



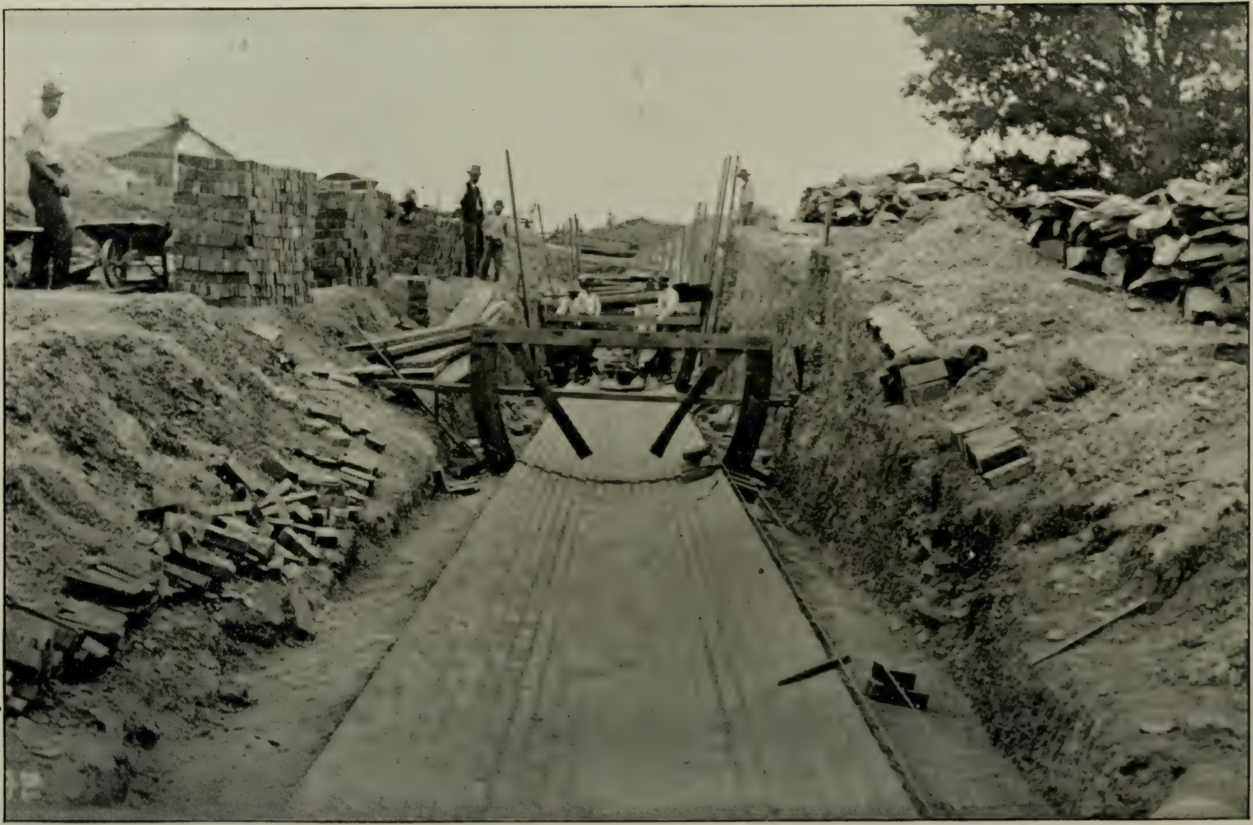
6.—VIEW SHOWING THE RIVETING TOGETHER OF TWO SECTIONS OF 38-INCH STEEL PIPE ON THE BANK OF THE TRENCH.



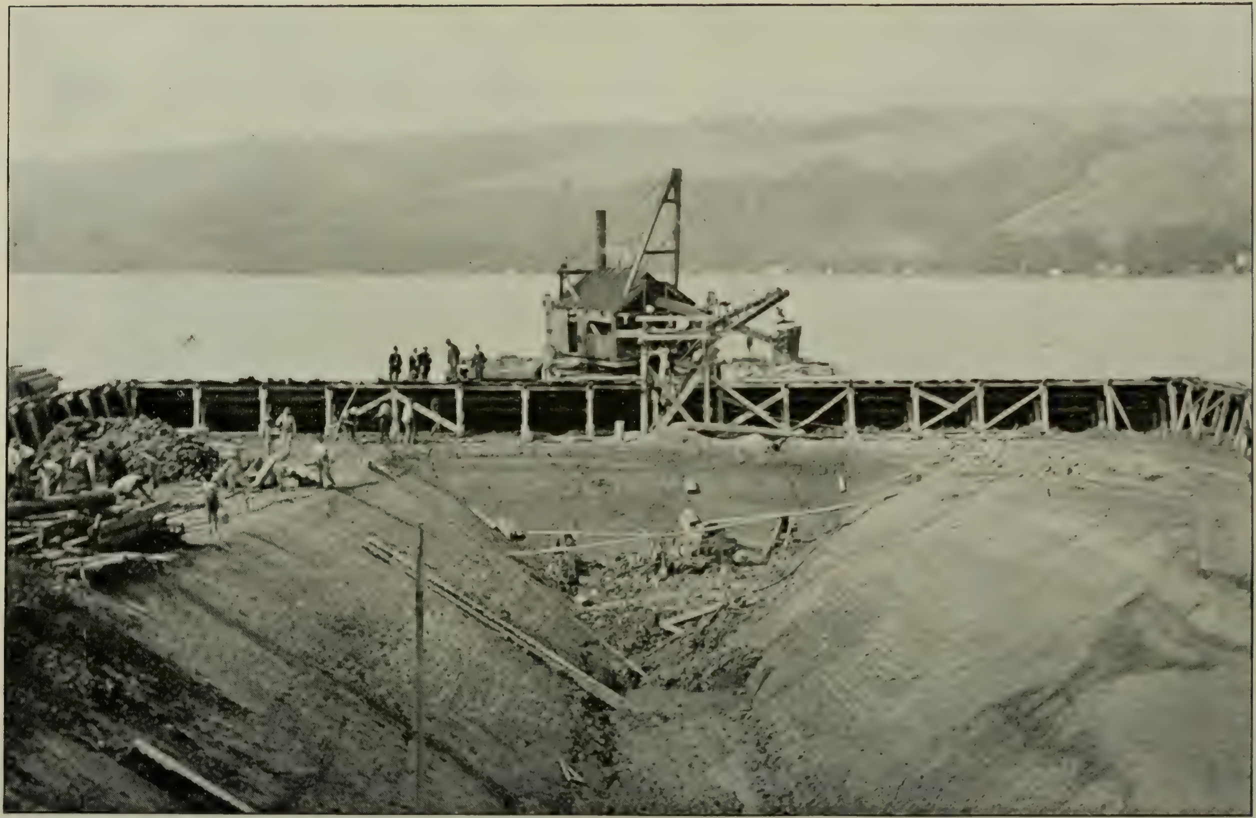
5,—VIEW SHOWING WORKMEN EXCAVATING THE TRENCH FOR THE 38-INCH STEEL PIPE.



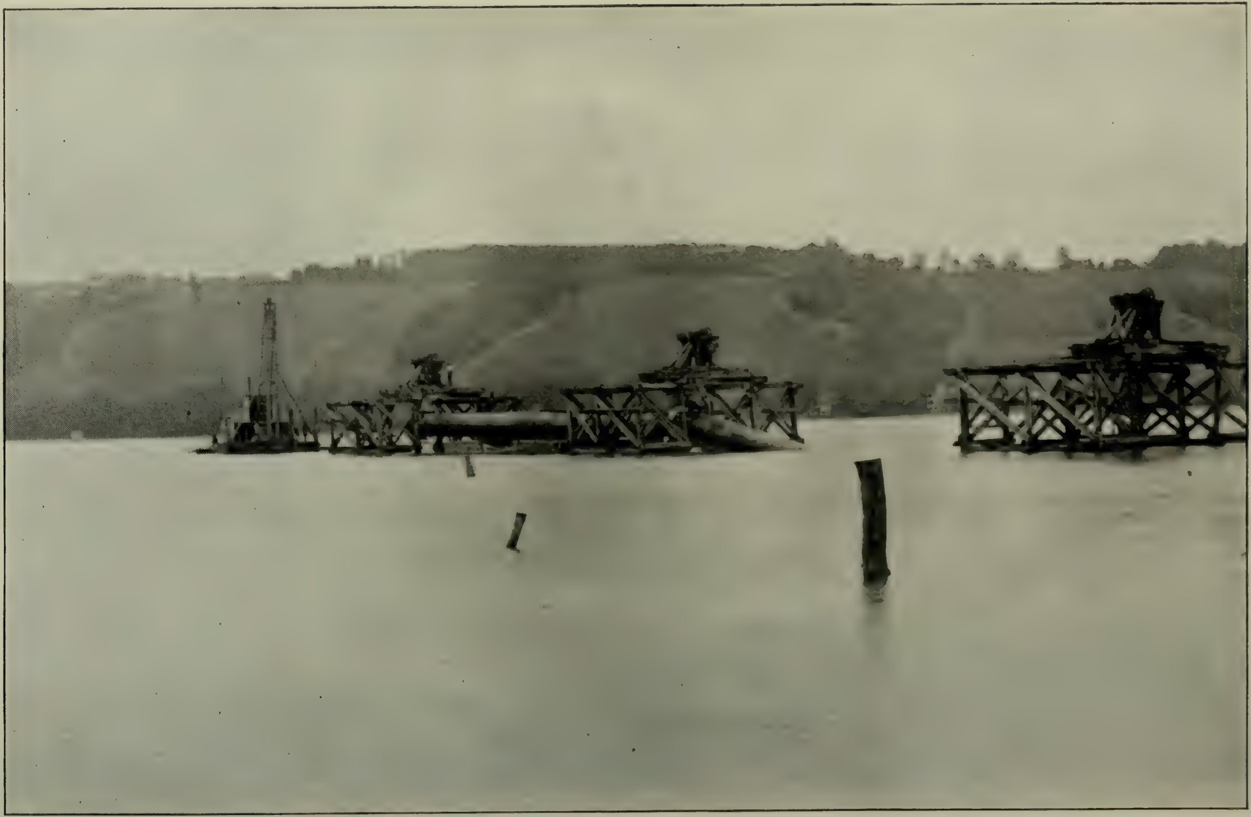
4.—VIEW OF FOUNDATION FOR OVERFLOW-CHAMBER AT END OF BRICK CONDUIT AND BEGINNING OF STEEL PIPE.



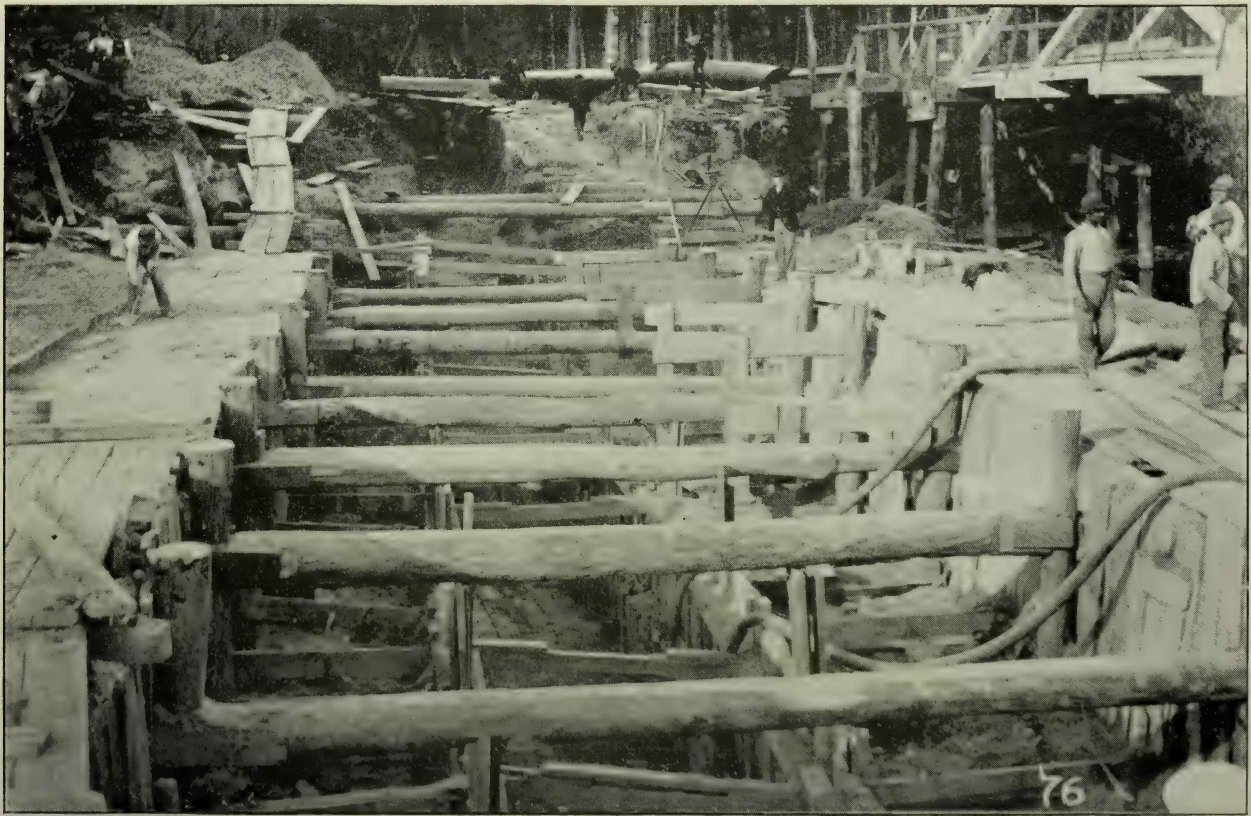
3.—VIEW SHOWING INVERT OF 6-FOOT BRICK CONDUIT LAID AND FORMS SET FOR THE SIDE WALLS.



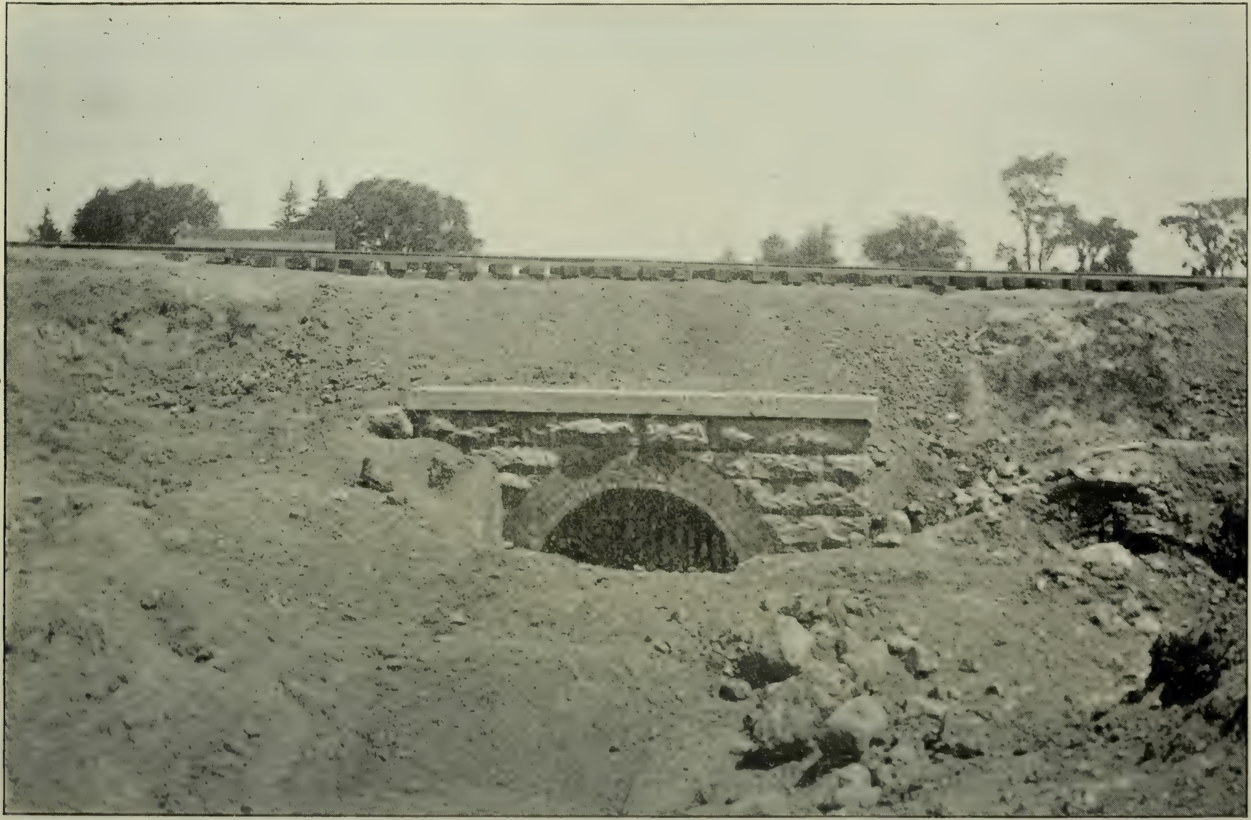
2.—VIEW SHOWING OPEN EXCAVATION FOR FIRST LONG SECTION OF 60-INCH STEEL INTAKE PIPE, ENCLOSED BY A TEMPORARY DAM IN SHALLOW WATER.



I.—VIEW SHOWING TEMPORARY PILE PLATFORMS IN LAKE AT INTAKE PIPE JOINTS, TOGETHER WITH PIPE AND WINCHES.



9.—VIEW SHOWING THE TIMBERING OF THE TRENCH FOR THE STEEL PIPE AT A CROSSING UNDER THE BED OF HONEOYE CREEK.



10.—VIEW SHOWING BRICK CULVERT BUILT FOR RECEIVING THE STEEL PIPE UNDER THE TRACKS OF THE WEST SHORE R. R.

THE PRESENT EUROPEAN PRACTICE IN THE DISPOSAL OF THE WASTES OF LARGE CITIES.[1]

JAMES H FUERTES, C.E. '83.

To those not conversant with the details of municipal administration, the thought may not have occurred, that one of the most difficult tasks imposed upon a city government is to provide the ways and means for rapidly removing from the community the wastes which are the result of the daily life of the people. When it is realized, however, that the average house in a suburban district produces from a ton-and-a-half to two-tons of refuse in a year, it is not difficult to understand that the disposal of the large quantity that would daily accumulate in a large city might present many difficulties.

In moderately small communities, in all countries, the almost universal practice is to have the matter carted to some uninhabited low territory and dumped upon the ground. But in large cities it is necessary to abandon this method, not only for sanitary reasons, but because, generally, suitable places cannot be found close enough to the center of population to make the method sufficiently cheap.

The refuse of cities consists of two classes of matter, organic and inorganic. The inorganic matter is comparatively stable in its character, but the organic matter is likely to decay or putrefy, thus creating various nuisances and dangers to health, and it therefore requires particular attention from a sanitary standpoint. The comparatively large amount of organic matter in city refuse has led inventors and investors to develop many schemes for its reduction, in order to recover its marketable ingredients.

One of the most marvelous works of nature is the transformation of dead organic matter into the food for living plants. The process goes on continually, and what is called putrefaction or decay is one of the evidences of this work. The transformation is brought about through the agency of the nitrifying organisms, microscopic forms of life which are everywhere present, and the rapidity of the process depends upon the chemical composition of the matter, the supply of oxygen, the temperature and the degree of moisture.

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[1] Lecture delivered by James H. Fuertes, M. Am. Soc. C.E., before the College of Civil Engineering of Cornell University.

The organic matter in refuse contains combinations of carbon, nitrogen, hydrogen, and oxygen ; during its decomposition, the carbon is oxidized, the nitrogen converted into ammonia and subsequently into nitric acid ; finally, mineral salts, which form plant food, are produced. These processes, in the free air, are accompanied by the evolution of gases and other products, with sometimes a very disagreeable odor. The animal tissues yield, among other compounds, certain ptomaines, the direct result of the action of microbes on the albuminoid molecules. These ptomaines are generally crystalizable compounds, all more or less poisonous, and they are more frequently than not the cause of poisoning from the eating of slightly decayed fish, meats and canned goods.

Cold storage will not render fit for consumption foods which have begun to spoil, because, although the process of putrefaction will temporarily cease under such conditions, the cold will not destroy the poisonous ptomaines which may accompany putrefactive processes. This is clearly proven by the numerous cases of ice-cream poisoning which have been recorded.

Putrefaction and decay are, therefore, most active in matter in which the carbon combines readily with the oxygen in the air, or in the moisture which must necessarily be present. They are also most active at temperatures which are most conducive to plant life ; that is, microbian life. At low temperatures they are checked ; at warm summer heat most active, and at higher temperatures they will again cease. Hence the necessity for the more frequent removal of refuse and garbage during the summer months than during the winter.

With the presence of moisture, the microbes of many specific diseases may find in these masses of decomposing organic matter suitable conditions for their sustenance, and possibly for their propagation ; such accumulations, therefore, may become centers of infection from which disease may spread. This is the feature of the disposal of refuse which, from a sanitary point of view, has greatest weight.

Should an epidemic of some very contagious disease visit a city, the removal of the refuse might become a difficult matter, because, being possibly infected, its removal to a tipping place would only transfer the danger to another locality, to be spread again by flies, other insects and birds which feed on carrion, as well as, probably, by the wind and dust. In such a case safety would lie in thorough disinfection, which can best be accomplished, with such large quantities of matter, by burning it. The conversion of the organic matter into useful compounds by the employment of high temperatures would satisfactorily purify that part of the refuse, but the other parts—the bottles, cans, boxes, cast-off apparel, vault and cess-pool cleanings and ashes,—amounting to perhaps 90% of the total wastes of a city, and which are perfectly capable of becoming infected and of transferring infection to other localities in various ways, would still require disinfection and removal.

The dumping of putrescible matter in low places to be afterwards occupied by buildings, cannot be too severely condemned. The piling of it upon the ground in the corners of yards or in open fields is, also, an evil to be forbidden, because of the dangers arising from the accumulations *per se*, as well as from the consequent seeding of the soil underneath the pile. The surface layers of damp soils impregnated with organic matter are very fertile beds for the growth of microbes, and among those which thrive under such conditions, and which are particularly dangerous to health, are the bacilli of anthrax, of tetanus, of malaria, of Asiatic cholera and of typhoid fever.

The wastes of cities, which may be designated by the generic term, Refuse, consist of various classes of matter, to each of which is given a specific name. These classes are, Garbage, Rubbish, Ashes, Night-soil, Street-sweepings, Dead Animals and Infected Materials.

Garbage consists of organic matter capable of infection ; such as vegetables, meat, bread and other scraps from kitchens, the wastes from markets and market gardens, and all other refuse of a vegetable or animal nature arising from the needs of domestic life.

Rubbish consists of old clothes, rags, papers, bottles, cans, pots, sticks, straw, boxes, and other similar cast-off and cast-out matter.

Ashes consist of the cinders and ashes from fires, and of the sweepings, dust, scraps of rags, papers, etc., from buildings.

Night-soil includes the contents of earth closets, middens, privies, cess-pools and other refuse of feculent origin.

Street-sweepings consist of horse droppings and dirt from the streets. The sweepings from paved streets are sometimes separated into these two classes of matter and each collected separately.

Dead animals include horses, cattle, dogs, cats, etc., gathered up by the scavenger, condemned meats and fish, and refuse from the slaughter houses.

RECEPTACLES FOR REFUSE.

In England, the refuse is generally stored upon the premises of the house-holder, either in a fixed ash-bin, or a movable receptacle. The ash-bin usually is a brick house, roofed and ventilated, with a capacity of about 20 cubic feet. It is obliged by law to be at least six feet distant from the nearest inhabited building. In 1890 an act legalized the use of an ash-tub with a capacity of about 2 cubic feet; enough to contain a week's accumulations. This was designed to encourage the disuse of the large ash-bins, which are excellent breeding grounds for various microbes, because the brick walls absorb and retain considerable quantities of moisture and are difficult to keep in a satisfactory sanitary condition. The smaller tubs, which can easily be carried on a man's shoulders, are usually made of galvanized iron and are provided with a tight-fitting cover. These permit of much greater cleanliness than the ash-bin, limit the amount of matter that can be stored, and therefore necessitate the more frequent collection of the refuse. In England this is a decided benefit because there it is the usual custom to throw the garbage into the ash-bins, instead of maintaining separate garbage pails.

In Hamburg, Berlin, Munich, Paris, Zurich, Florence, and, in fact, in most of the large cities, the movable galvanized iron receptacle is used, and in very few is the pit or bin countenanced.

THE COLLECTION OF REFUSE.

The refuse is usually collected by the municipality or local authority. Sometimes, however, the contract system is employed, and in some small cities each householder is obliged to provide for the removal of his own refuse. Until recently this latter method was employed by the city of York, England, which at that time had a population of about 67,000.

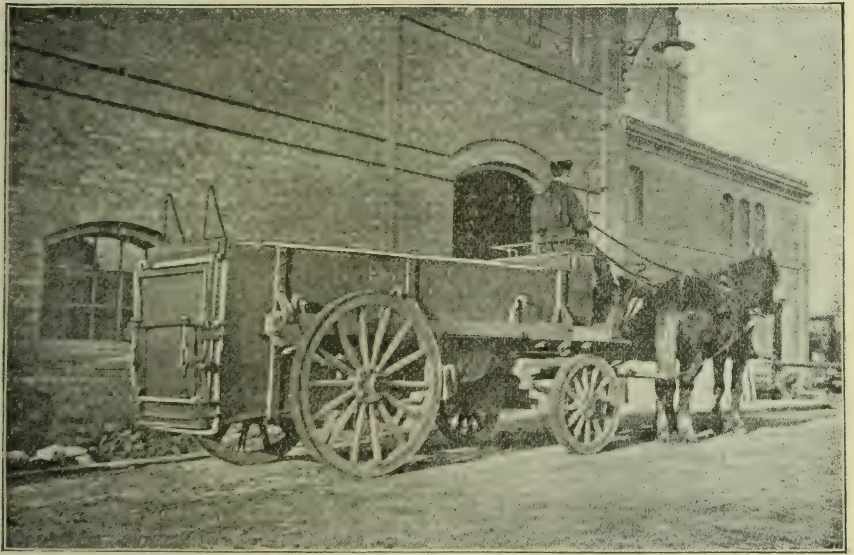
In some of the towns in the South of England, the refuse is collected by licensed men with push carts. They go from street to street and the jangling of the bells on their carts is the signal to the householder who sends out his refuse in portable receptacles.

In rural and suburban districts, collections are quite generally made upon receipt of notice from the householders.

Sometimes, in districts of a more urban character, collections are made at certain intervals, weekly or fortnightly, with special collections upon receipt of notice. This is an improvement upon the method previously mentioned as it insures a certain amount of regularity and system in collecting. The most usual custom in the large cities is for the carts to call periodically from house to house to collect the refuse. Sometimes collections are made daily and sometimes at longer intervals. In nearly all the large cities the municipalities make the collections with their own employees and teams. Mr. John Price, A.M. Inst. C.E, found, upon inquiry, that 70 out of 85 English cities collect their own refuse.

It is the quite general experience in all cities that the success of the contract system depends largely upon the basis of payment, and neither a contract for a fixed sum nor for a price per load will always insure that careful attention to details that is necessary to do such work efficiently and in a manner satisfactory to householders. Most cities have, therefore, found it better and frequently cheaper to have full control of the refuse collection.

The vehicles for the removal of the refuse are generally large wooden wagons, provided with hinged tops, or else covered, when loaded, with canvas laced down tight. The best vehicles, however, are iron or steel, two or four wheeled, with sectional hinged covers, and so constructed as to allow dumping. The wagons used in Hamburg are of iron, and are provided with hooks on the sides so that the bed and its load may be lifted from the running gear by an over-head crane, and the contents dumped on the platform above the incinerating furnaces.



Hamburg Garbage Wagon.[1]

The collections are generally made during the night hours; between 10 P. M. and 9 A. M. The wagons visit each house and remove the ashes, wash and disinfect the bins, if necessary, remove the pails from closets and replace them with clean ones, and then take the collections to the place provided for their disposal.

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[1] The cuts of the Hamburg Garbage Destructor Plant were kindly loaned by Mr. Rudolph Hering, M. Am. Soc. C. E., New York.

THE DISPOSAL OF REFUSE.

Barging the Refuse to Sea. This practice is possible, of course, only at sea ports or at cities upon navigable waters not far distant from the open sea. The method is extensively employed at Liverpool and Dublin as to refuse, and at London for the disposal of the sludge from the sewage precipitation works at Barking and Crossness. Liverpool, however, has a large plant for burning its refuse, and is considering the extension of the system to various parts of the city.

The principal objections made to dumping refuse into the sea are the danger of floating matter being washed back upon the shore by winds and currents, and the dangers, from the tin cans and broken bottles, to the trawling nets of the fishermen. There is also the possibility of continued rough weather making sea trips impracticable for small boats. However, all these are questions which local circumstances will decide in each case, and such a method in some localities might be wholly unobjectionable. The expense of sea disposal at Liverpool, where the dumping grounds are twenty-four miles from the landing stage, is given at 37 cents per ton. Until 1891 about 70% of all their refuse was disposed of in this way.

The sludge from the London sewage precipitation works, which averages 91% water and amounted to over 2,000,000 tons last year, is taken to sea in five steamers especially designed for that purpose, with a capacity of about 1,000 tons each. The dumping grounds for this liquid mud are about 50 miles from the outfalls of the sewers, and the cost of transporting it to sea after it is loaded into the vessels averages about 9 to 10 cents per ton.

Tipping. Dumping refuse from tips into disused quarries, clay pits, low grounds, and wet places, is extensively practiced. It is never advocated as a desirable method, but is of necessity frequently unavoidable. Generally, only the rubbish, ashes and sweepings are tipped, the garbage being disposed of in other ways. Frequently the garbage is cooked in large cauldrons and the grease which rises to the surface is skimmed off and disposed of to soap makers, while the remainder is fed to swine and cattle or else rotted and mixed with horse droppings, street sweepings, etc., to form a saleable manure. Dead animals are sometimes buried and infected materials destroyed in bonfires.

These primitive methods are widely distant from the demands of proper sanitation, and yet they are practiced to a certain extent in nearly all large cities. Manchester, Glasgow, Edinburgh, Bristol, Leicester, Newcastle and many other cities still tip a portion of their refuse. It is but just, however, to say that nearly all of these cities are at the present time encouraging other methods of disposal, in line with the more satisfactory processes developed in recent years.

Utilization. The utilization of the valuable ingredients of the refuse is practiced in various degrees in nearly all cities, because there are several classes of matter, easily separable from the mass, which have a marketable or recoverable value. From the refuse it is possible to separate: Fuel, consisting of cinders and coal from ashes, sticks, papers, rags and other burnable matter; old iron and copper; bottles; rubber scraps: papers and rags; ashes and clinkers which are used in foundations under cellars and side-walks, as filling in fire-proof floors, in loosening up clayey soils and in filling up low places. In addition to these dry materials are: Night-soil, valuable as a fertilizer; manure, consisting of rotten garbage, horse-droppings, fish refuse, sewage sludge, fine dust from screenings and street-sweepings; finally, dead animals and condemned meats which are valuable for their grease.

In cities where attempts are made at the separation of the refuse into various classes, the sorting is done partly by mechanical appliances and partly by boys, men and women.

Most utilization processes require a considerable amount of power to operate the necessary machinery, and hence a fair fuel is desirable for steam purposes. This is usually refuse from which the dust has been removed by sifting and the unburnable materials removed by hand.

In Manchester the refuse is separated into three classes of material at one screening: fuel, consisting of coal cinders, papers, sticks, etc., which fall through a chute to the boiler floor; dust, which is caught in a cloth sail below the screen and sent through a chute into railway cars; and old iron, copper, bottles, etc., which are removed from the foot of the screen by a man. The two screens at the Holt Town works each separate about 175 tons of refuse per day.

At Glasgow the separation is effected by dumping the mixed refuse into a hopper from which it passes to a slowly revolving riddle. The coal and dust fall through and the large matters work out of the end and fall upon a wide belt conveyor which extends to the destructor room. Boys along the conveyor remove unburnable matter like bottles, iron, etc., and the rest is burned in furnaces. The dust and cinders which fall through the riddle are separated by an ingenious arrangement, by which the coal goes to the boiler rooms and the dust to railroad cars below.

At Paddington the refuse is sorted and screened, and the cinders and ashes are sold to farmers.

At Chelsea the screening is done mechanically, and, by the aid of an air blast, the paper is separated from the other refuse, and the dust is blown into the ash pit of a boiler. The screenings are sorted by hand for rubber, boots, shoes, coal and metals, and the other organic refuse is sent to a mill and ground into manure. The paper refuse is sterilized by hot steam and reconverted into brown paper, having a value of from $35.00 to $40.00 per ton. Coal and cinders are pressed into fuel blocks.

Night-soil is generally converted into a fertilizer. At Manchester it is made into a concentrated fertilizer by drying it in steam jacketed cylinders; at Rochdale it is made into a poudrette by a blast of hot air passing over and through it in a revolving cylinder, and the resulting dried mud is ground in mills. In both processes certain chemicals are added to fix the ammonia. These fertilizers have a value of from $15.00 to $30.00 per ton, and considerable quantities are sold.

The most general method, however, of disposing of night-soil is to mix it with street-sweepings, ground fish refuse, ashes, or with dessicated or mashed garbage. This is sold to farmers to enrich their land, or plowed in on lands belonging to the corporation.

These processes are more extensively employed at Manchester than at any other city of Great Britain. As a newly constructed system of sewerage and sewage disposal works has been installed, and as the introduction of water-closets is gradually driving the disgusting pan system out of use, the manufacture of this kind of fertilizer will soon be with them a thing of the past. In spite of the revenue derived from the sale of the products of its reduction works, the City Engineer estimates that the introduction of sewers will effect a great annual saving in expense to the cleansing department, and will also exert a beneficial effect upon the healthfulness of the city.

Old cast-iron, old copper and bottles have a ready sale and their handling forms a considerable industry. Old cans and soldered wares, however, are usually difficult to dispose of; generally they are a waste so far as the corporation is concerned, but poor men can make wages by melting the solder out of them. They can be disposed of sometimes to copper houses, where they are used to precipitate the copper from solutions.

Dead animals and condemned meats have a value for the grease and phosphates. The grease is converted into wagon grease, engine grease, lard-oil and soap, and the residue into a fertilizer. Manchester utilizes her animal wastes in this way and derives considerable revenue from it by the saving of expense which would otherwise have to be incurred in purchasing such articles.

The disposal of sludge from sewage precipitation plants in most cities having such plants is a very difficult problem. At Wimbledon, England, it is disposed of by pressing in cakes and selling it at 25 cents per ton ; such as cannot be sold is spread upon the farm. Its value as a fertilizer being very small, it is planned to burn it in a destructor. At Ealing, England, the sludge is mixed with refuse and burned ; at Southampton it can, also, be disposed of, in part, in the same way. At Friern Barnet it is pumped upon grass lands and allowed to dry and soak away ; it generally kills out all the vegetation where it lies. However, the most usual method is to press the sludge into cakes and dispose of it as a fertilizer, sometimes receiving a small compensation for it, sometimes paying for its removal and sometimes burying or tipping it. Some special processes convert it into a fertilizer by the addition of other matters, as is done in the A. B. C. process at Kingston-on-the-Thames.

At Frankfort-on-the-Main, the sludge is pumped into large open shallow ponds and allowed to evaporate and dry out, and is disposed of to farmers and used on the garden lands belonging to the city. The farmers have tank wagons into which they pump this semi-liquid black mud, and seem to find it sufficiently profitable to warrant them in carrying it away during seasons when it can be used.

At some cities the sewage sludge is pumped into pies and mixed with street sweepings to make a fertilizer, as is done at Southampton, or the emptyings of catch basins, street sweepings and refuse are mixed, as at Liverpool. However, these are only the expedients adopted at different cities because of the necessity of doing something, and that something is generally the outgrowth of the conditions and possibilities at hand.

The horse-droppings on the streets are generally swept up by boys or men and put in special receptacles. Sometimes they are gathered up by the corporation, as at Manchester and London, and sold to farmers. Sometimes they are gathered up by farmers' boys without charge to or from the corporation, as at Leeds. They always have a value and are saved in nearly all European cities. It is the general custom in Italy for the wagons traveling between the towns and the market gardens and small farms, to have special buckets in which to deposit the horse-droppings found on the country roads ; and it is no uncommon thing to see women and children with sacks and buckets gathering them up in the road and carrying them to enrich their little kitchen gardens, so great is the economy necessary in these thickly settled old countries.

At Leeds the machine sweepings from the paved streets are piled up in certain districts and used as needed as a binding material for newly made macadamized streets.

Destruction of Refuse by Fire. All these methods for the disposal and utilization of the refuse of towns are open to the very important objection, from a sanitary point of view, that should one of the violent epidemics of cholera, yellow fever, or other plague take root in or threaten a city, it could find no consumers for its products ; no purchasers would take its fertilizers or its refuse, and the city would be left with a rapidly accumulating mass of decomposable matter ripe for infection. This was the predicament in which the city of Hamburg found itself in 1892 when visited by cholera, and was the principal cause for the adoption of the policy of destroying all its refuse by fire.

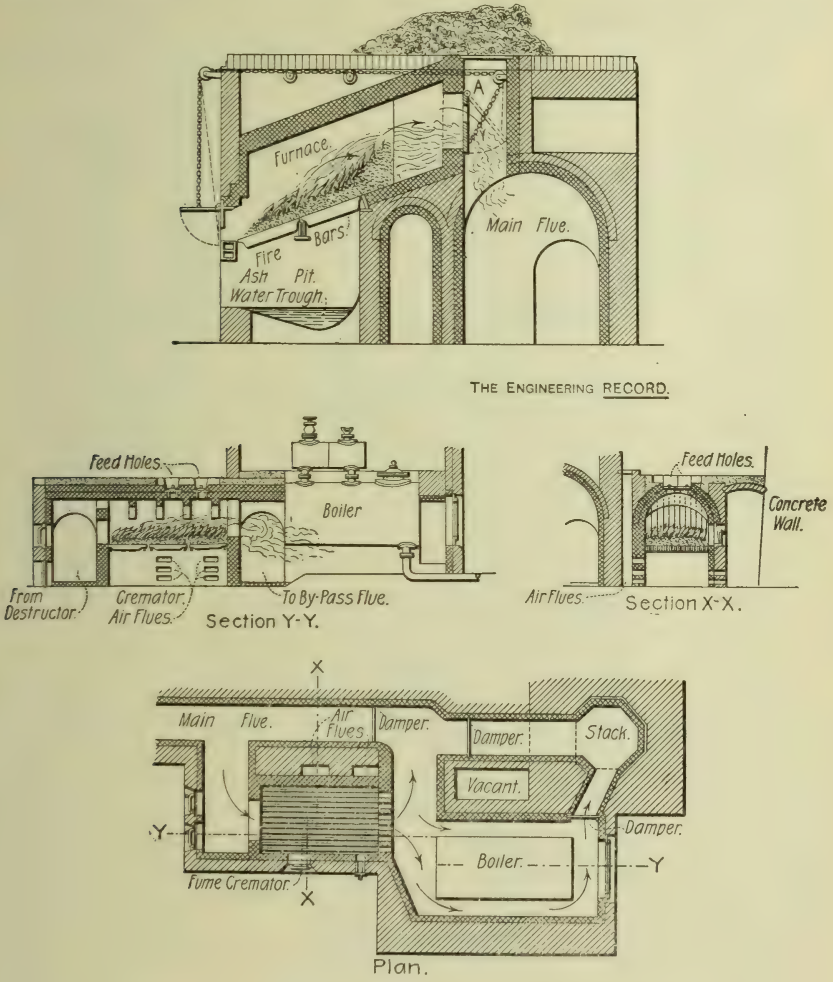
England is the pioneer in the field of refuse destruction by fire, and there are now over five hundred cells or furnaces in operation in the various cities. Her example is being followed by the continental and several American cities, and inquiries are being made by cities in Russia, Switzerland, Australia, Africa and Brazil. It will not be possible to describe in detail the construction of the various makes of destructors in use abroad, all of which are patented and only differ from each other frequently in very small matters.

The refuse collected in towns has a varying composition, depending upon the portion of the town from which it is taken, the time of the year, the meteorological conditions and the habits of the people in different countries.

The refuse of English cities contains more combustible matter than the refuse of German cities and the refuse of the latter contain more combustible matter than does that of Italian cities. Refuse from the rich quarters of a town will contain more combustible matter than that from the poor quarters; refuse collected in winter will contain a much larger proportion of coal and cinders than that collected in summer ; refuse from agricultural and commercial districts will contain much less fuel than that from manufacturing districts ; and in rainy weather the refuse will contain more free water than in dry. All these variables affect the burning of the refuse and point to one of the most important features in destructor construction. Combustion is a purely chemical process by which organic matter combines rapidly with the oxygen of the air. If, therefore, the composition of the refuse varies from time to time, the quantity of air consumed for perfect combustion will vary, and means for regulating this supply are necessary.

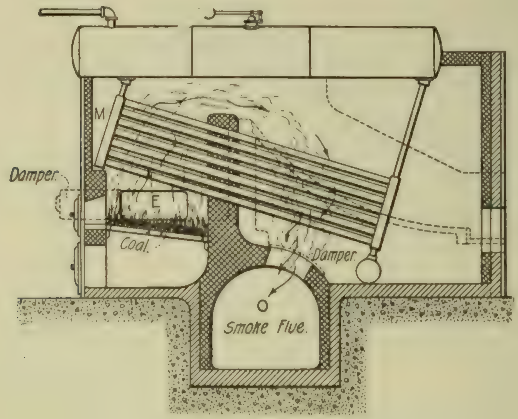
In most extant destructors it seems to have been the object to provide as strong a draft as possible by the construction of a very tall chimney, by a blast of air, a steam jet or by combinations of these appliances. This frequently has resulted in producing a draft so strong that the fire has been pulled apart, causing dust and ashes to be lifted out and spread over the surrounding premises, to the annoyance of property holders and the discredit of the system.

Since the burning of refuse generates a certain amount of heat in excess of that which is necessary to evaporate the contained water and to consume the vapors and gases arising from the decomposition of the organic matter, the attempt is usually made to utilize this heat and



Ealing, Eng., Garbage Destructor and Fume Cremator.

transform it into power, to be applied according to the needs of the plant or the opportunities offered by the local surroundings. In some plants the effort is made to develop the maximum amount of power, and the effectiveness of the destructor in burning the gases generally suffers thereby. In other plants the effectiveness of the destructor is first considered and the power producing properties made of secondary importance. Generally speaking, the best conditions for power development are opposed to those of nuisance prevention. It is, however, a fact that there are some plants which burn refuse without the addition of any fuel, from which from 4 to 6 H. P. per cell is derived without



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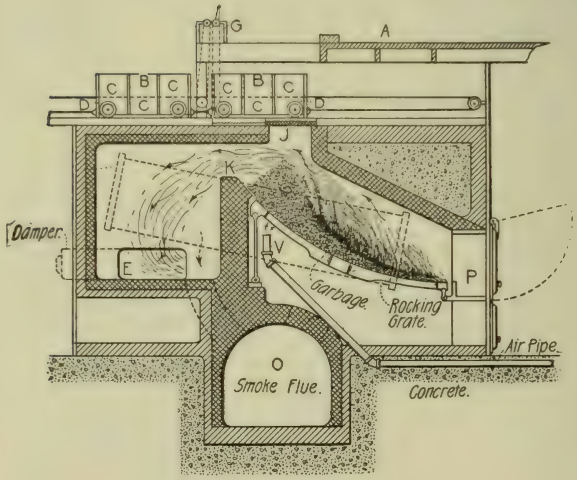
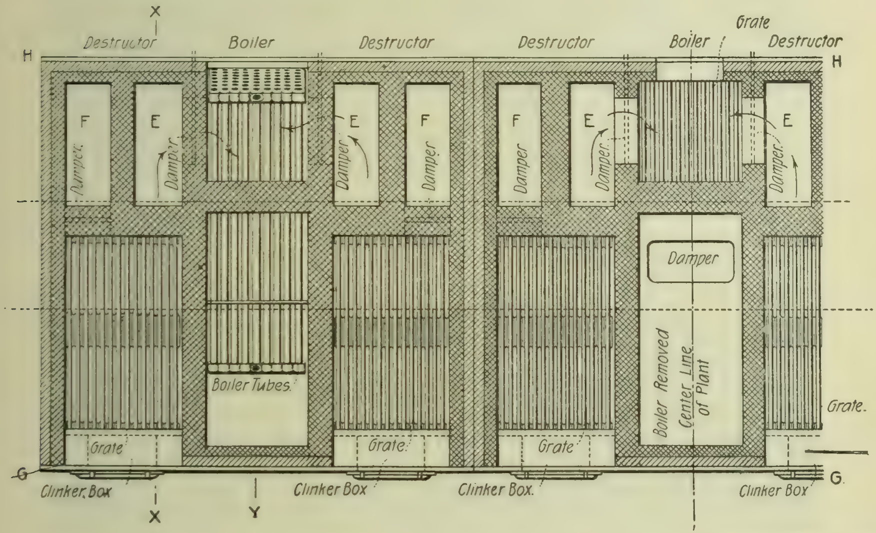


Section X-X

THE ENGINEERING RECORD.

Cambridge Refuse Destructors.

the creation of a nuisance. If a second fire is maintained with fuel, so that gases not burned in the destructor can be passed over it and destroyed before escaping into the chimney, then a greater amount of power can be developed without the creation of a nuisance, because the boilers can be brought closer to the furnaces, relying on the second fire to burn the gasses cooled off by contact with the comparatively cold surfaces of the boiler.



The Engineering Record.

Cambridge Refuse Destructors.

In nearly all the English destructors, the heat is used to generate steam in boilers placed in the flue near the base of the chimney. In some later plants, notably at Cambridge, and a proposed extension of the Southampton works, a different arrangement is proposed by which there will be one boiler to each two destructor cells, with special fires to cremate the gases before they pass up the chimneys. Such arrangements as these can undoubtedly be made to develop much power, and, also, under proper conditions, to do so without the production of a nuisance.

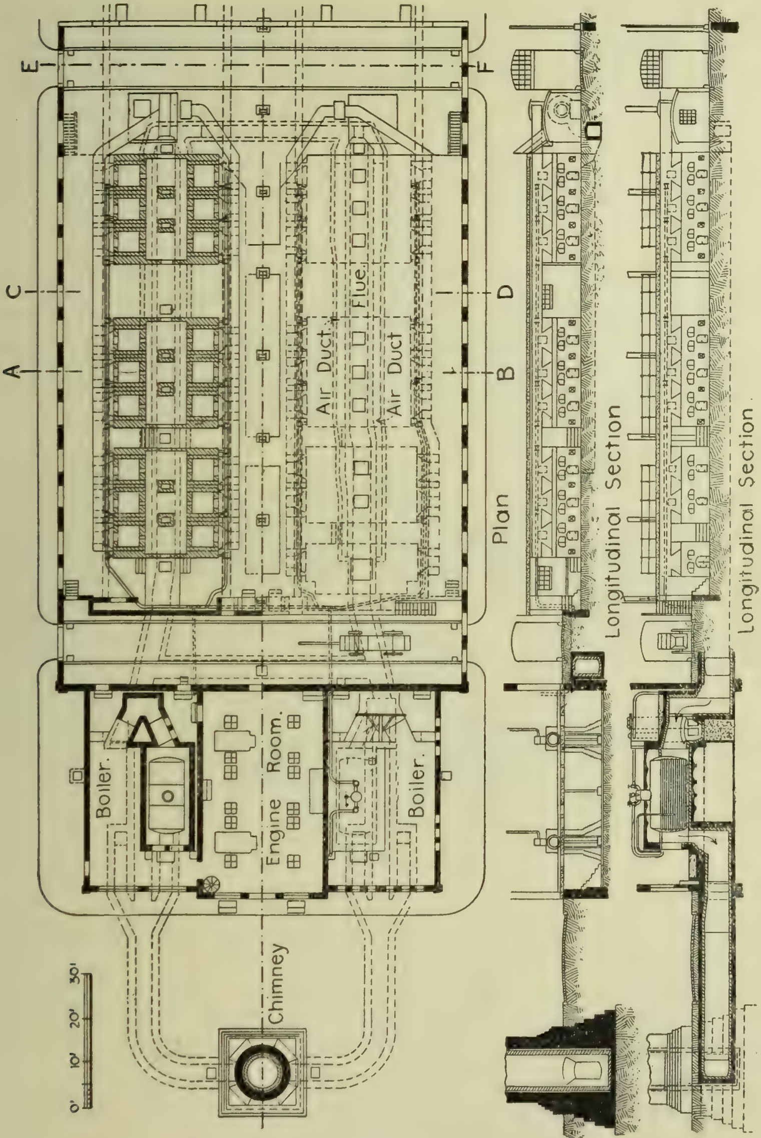
Destructors consist, essentially, of an inclined grate in a fire-brick furnace with a fire-brick hearth, back of the grates, upon which the refuse is dropped and allowed to dry out somewhat before it is pulled down onto the live fire on the bars. The different destructors introduce variety in the method of leading the burning gases out of this furnace to the main flue and thence to the chimney.

Some take them out over a bridge wall back of the hearth and allow them to pass over the drying refuse; others take them out of flues in the side of the furnace near the back of the fire; others near the front of the fire and others through openings in the roof of the furnace directly over the hottest part of the fire.

Hamburg Garbage Destructors.

The methods of charging the fires with refuse are peculiar to the different makes of destructors. In some, the refuse is dropped upon the hearth in a charge of fixed amount; in others it is fed intermittently by hand with long bars; and in others it is fed continuously, either by hand or by special patented charging appliances. In most of the furnaces the grate bars are fixed and the stoking and clinkering is done by hand. There are several patented automatic stoking bars, and in two makes of destructors their motion is depended on to push the refuse forward on the bars, break up and eject the clinker, and thus save labor. In most places they are not regarded with favor, principally for the following reasons: their motion permits the fine dust to fall through the grates without burning it; if the attendants do not watch them carefully, the clinkers burn fast and thus require hand stoking in any event; and as a practical result of trials, they have not demonstrated their ability in destructor work to save much expense.

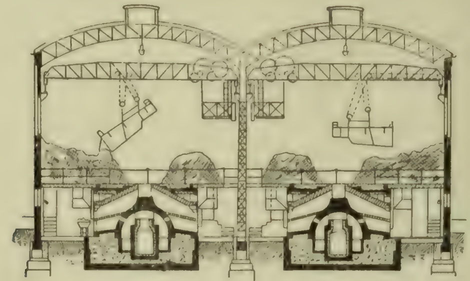
Self-feeding furnaces are theoretically desirable, but with fuel of such variable composition as town refuse the rate of feed must be suited to the character of the refuse and automatic stoking has not yet come into



Hamburg Garbage Destructors.

general use. The most recent plants are being made with fixed grate-bars or with bars that can be shaken occasionally by hand. With screened refuse, movable grates would undoubtedly give more satisfactory results than with ordinary refuse.

The burning of refuse requires skill and experience on the part of the firemen. It is necessary to have a hot fire and to rake the fresh refuse upon it in such a way as not to smother it down and reduce the temperature, for that causes the escape of foul gases. The operation of clinkering requires, also, great care and judgment, because, when being done, a large quantity of cold air is admitted to the furnace, the continuity of the fire is broken and the temperature reduced by the removal of the clinker.



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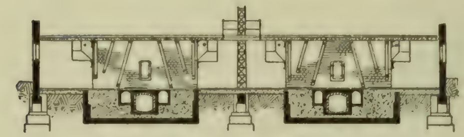
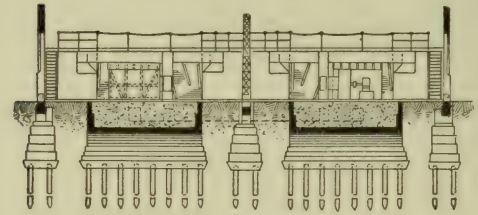


Cross-Section on Line C-D.



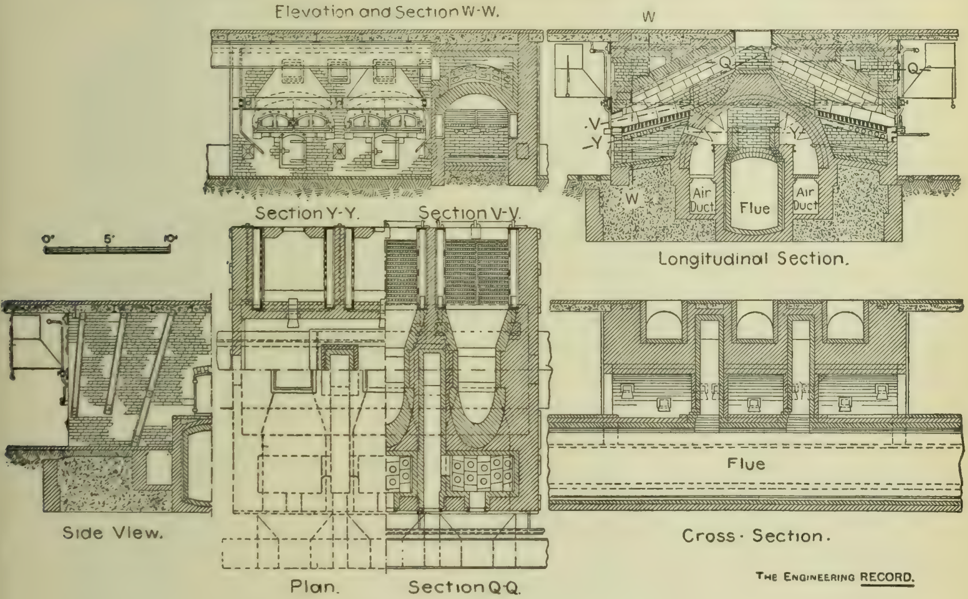
Cross-Section on Line E-F.

THE ENGINEERING RECORD.

Hamburg Garbage Destructors.

In many furnaces special dampers and baffle plates are provided for shutting off a cell which is being clinkered and fired, thus preventing, as much as possible, the admission of cold air into the main flue, which by lowering the temperature would produce nuisances.

In general the requirements are, that all features of the construction and operation of the furnaces should reduce to a minimum the danger of the lowering of the temperature in the flues and cells. These are the principal points upon which the efficiency of refuse destructors as built in Europe depend. Where they are considered, high efficiency and no nuisance result, but often small details improperly designed, or incompetent management, may destroy the possibilities of an otherwise good furnace. It is much more of an art to fire and clinker a refuse destructor than to handle the fire of a steam boiler or locomotive, and yet the men who are entrusted with this work, upon which the success of the system may depend, are, more often than not, poorly paid and inexperienced.



THE ENGINEERING RECORD.

Hamburg Garbage Destructors.

I have in mind the case of one city which has been a defendant in a succession of suits, and in which a very excellent plant has been declared a nuisance. This was brought about principally, because its management, after completion, was turned over by the corporation to the scavenging department. There it was entirely removed from the supervision of the able engineer who designed it, and under whose management and watchfulness it would probably have yielded very different results.

Usually, cities having plants for refuse destruction send all their collections to them without separation, excepting perhaps the street sweepings, which have a value and ready sale.

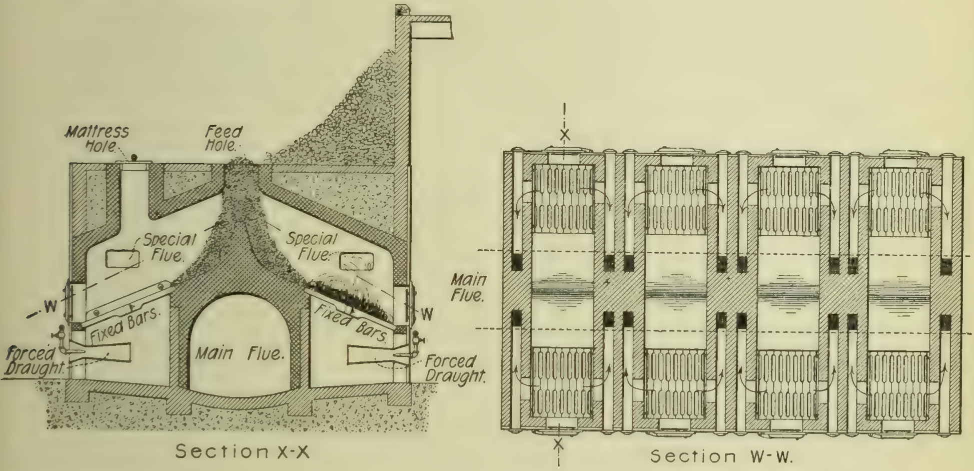
In most of the cities of Great Britain, in Hamburg, Berlin, Zurich, and many other cities, it is the custom to deposit all the garbage, ashes and house sweepings in the same receptacle, and not to keep separate tubs for the different classes of matter. This is favorable for the process of burning, as the addition of the unburned coal and cinders aids in the evaporation of the moisture in the garbage.

The relative economy of different nations is reflected in the character of their refuse. For instance, the refuse of English cities contains so much coal that at no place, to my knowledge, is additional fuel required to burn it, while at Hamburg it burns with difficulty, and at Berlin fuel has been found necessary in the process to the extent of nearly 10%, on the average.

Perhaps the most unfavorable refuse for burning is that at Ealing, England, with which the sewage sludge is mixed. The mixed sludge and refuse, as they are taken to the top of the destructor, are in a saturated condition, as only the free water has been allowed to drain out. As it is fed to the furnaces it is mixed with twice its bulk of the ordinary refuse, which contains in itself perhaps about 25% its weight of water. In spite of this very high percentage of water, and the fact that the sludge proper contains scarcely 18% of burnable material, the heat from the burning matter is sufficient to evaporate the water in the fresh refuse, consume the combustible portions, and convert the residue into a hard vitreous clinker. The quality of this refuse makes the daily capacity of the cells low as compared with those of many other cities; it also reduces the power obtainable from the heat and makes necessary the maintenance of a second fire over which the burning gases are passed to destroy the odors.

Moisture and dust are the two greatest obstacles to successful refuse burning. The one requires heat for its evaporation, and the other chokes the draft, absorbs much heat while generating practically none, and is frequently carried out the chimney by too strong a draft. But as the burning of refuse would fall short of its aim as a sanitary measure did it not include the destruction of all refuse of every kind, these unfavorable elements must be considered, and with a well designed and properly operated destructor there is no practical difficulty that cannot be, and I may say has not been, met.

The disposal of town refuse must be at a charge in any event. There is no city in the world where it is otherwise, and the various methods devised for utilization are but expedients to reduce this charge. In the English cities such methods are growing less and less in favor, because the profits therefrom are not large, and there still remains after separating the valuable ingredients, something like 90% of the bulk of the original refuse to be disposed of, and which, if disposed of in a satisfactory way, should, excepting the ashes, be disinfected. Added to this is the fact that tipping places for ashes and dust, in time, become scarce within reasonable distances of cities; and that, although there is frequently a good demand for clean cinders and clinkers for building purposes, when they are mixed with house refuse they are valueless, and thus the bulk of the refuse, as stated above, amounts to about 90% of the collections.



Section X-X

Section W-W.

THE ENGINEERING RECORD.

Refuse Destructors at Leeds.

When refuse is burned in a destructor it is reduced to a clinker amounting usually to from 10% to 33% of the weight of the original mass. This clinker is of value as filling and roadmaking material, and when ground up makes good mortar and concrete. In the English cities, almost without exception, it is disposed of in these ways, without difficulty and at a profit.

The power generated by the heat is utilized, generally, to drive the machinery of the plant, and at night to light the plant by electricity. Recently it has been utilized, in a few cities, in connection with the electric installations of the towns, for which it is sufficient to furnish power for the day load, thus reducing the cost of the night runs. The possibilities of destructors as power producers have not yet been reached. One plant recently installed has developed nearly four times the power per ton of refuse consumed that has, heretofore, been considered practicably attainable.

As a short summary of a considerable amount of data gathered from many towns and cities, the following items regarding destructors will indicate the English practice and the results obtained.

One cell is generally provided for 8,000 people; varying from 3,000 to 20,000, according to local circumstances. The usual grate area provided is about 25 square feet to each cell. The average cost is about $2,000 per cell in moderately small plants, and decreases to about $1,000 per cell in large ones, including the cost of chimney. A fair and attainable consumption of unscreened refuse is from 7 to 8 tons per cell per day, when burning at economical rates, and from 10 to 27 tons as a maximum with strong forced draught and very frequent clinkering. At such great rates the cost of burning is very much more per ton than at the lower average rate.

The burned refuse reduces to a hard vitreous clinker of about one-quarter the weight of the original refuse, and is used in mortar, concrete, street sanding, sidewalk paving-slabs and road making. Ordinarily the working value of one cell is about 6 to 7 H. P. continuously with thermal storage, varying from 4½ to about 30 H. P. in certain plants designed specially for power development. The steam is used for operating mortar mills, fodder choppers, air compressors, sewage pumping machinery, and as an aid in electric installations. It is also used in steam disinfecting plants, public laundries and bath houses.

The cost of burning the refuse after its delivery at the destructors is usually about 37 to 50 cents per ton, but frequently is less than 25 cents. In some cities the sale or use of the products of the combustion will make the net cost of burning as low as 15 cents per ton. These figures refer, of course, to the English cities. Such as I have for the American practice are much higher. This may be due, somewhat, to the different conditions of society, but it may be, also, principally chargeable to the difference in methods employed.

UNSTEADY FLOW OF WATER IN PIPES—WATER RAM.

BY IRVING P. CHURCH, C.E., '73.

(Including abstracts from a thesis by Mr. B. F. Latting, C.E., '94.)

To the April and May issues of the Journal of the Franklin Institute for 1890 the writer contributed an article entitled "Water Ram in Pipes", containing a theoretical treatment of certain cases of unsteady motion of water in a pipe, as due to the gradual closing of a valve at the discharging end ; with several numerical applications.

In 1894, Mr. B. F. Latting, of the Senior class in Civil Engineering at Cornell University, took up the subject for a graduating thesis, and besides making some further developments in the theory, performed experiments with an eight-inch pipe some 2300 feet in length under a head of 300 feet. These experiments will be described later.

It is the purpose of the present paper, after re-stating briefly the theory of the original article, in a form adapted to immediate purposes, to give an account of Mr. Latting's further steps in applying the theory and of the experiments above mentioned ; and also to present some later theoretical developments by the present writer, with numerical illustrations.

Fig. 1 shows a vertical and longitudinal section of a long level pipe terminating at the discharging end in a short pipe or nozzle, furnished with a valve-gate. Through the pipe and nozzle, with full sections, water is flowing from the tank or reservoir, R ; and it will be assumed that the motion is *unsteady* ; that is, that the velocity of the long cylinder, or prism, of water in the pipe is accelerating, or retarding. For example, if the whole body of water were originally at rest and the nozzle instantaneously opened to its full extent, the water in the pipe would gradually acquire motion and its velocity become accelerated ; until, after a greater or less period of time, depending on the length of the pipe, the flow would become practically steady, the velocity having reached a maximum (velocity of steady flow). (According to strict theory, however, it would require an infinite time for the flow to become steady.) Again, after a steady flow had been in progress for some time, suppose the valve-gate at the nozzle to be instantaneously moved far enough to reduce the sectional area of the issuing jet (at contracted section) to only a fraction of its former value ; the water in the pipe would gradually be retarded in velocity, until after a time the flow would again become steady, for these new conditions. As a third case, let the valve gate be moved gradually, while opening or closing; then during the progress of its motion the velocity of the water in the pipe would be accelerated, or retarded, according to circumstances; while the pressure on the inside of the pipe at m, near the lower end, would fluctuate. Our problem now is to trace the fluctuation of this pressure, when an unsteady flow is caused by a gradual motion of the valve; also to find the values, at different stages, of the other variable quantities concerned.

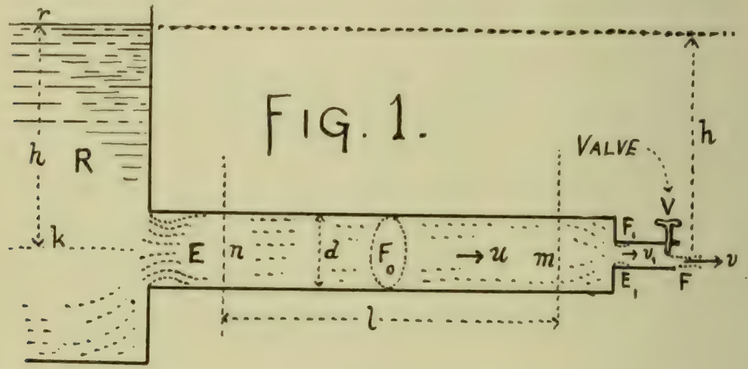


FIG. 1.

Referring again to Fig. 1, let l denote the length of the pipe, d its diameter, F_0 its sectional area, $= \frac{1}{4}\pi d^2$; and u the velocity of the water in it at any instant (*i. e.*, u is variable). Let F_1 be the sectional area of the short cylindrical pipe forming the nozzle, (both F_0 and F_1 are constant); and F that of the jet (*i. e.*, in its contracted section where the filaments are parallel and under atmospheric pressure) which issues into the air from under the edge of the valve-gate. F varies, as the valve-gate moves. Let the ratio $\frac{F}{F_0}$ be denoted by x, and the velocity in the jet by v, so that we have $v = \frac{u}{x}$ (from the equation of continuity, $Fv = F_0 u$); x_1 will denote the constant ratio $F_1 : F_0$. The vertical distance from reservoir surface to the nozzle is the "head," h, which in Fig. 1 also $= \overline{rk}$, the pipe being supposed level in the case of that figure. (Even if the pipe were inclined, the corresponding analysis would still lead to eqs. (5) and (6), to be established presently, so long as h represents the head of the nozzle.)

The position m in the pipe is taken just up-stream from the nozzle, but not so near but that the particles of water in the cross-section have practically a common velocity and are under a common pressure at any instant; similarly, position n is just down-stream from the internal contraction, at E, occurring at the entrance of the pipe. The internal fluid pressures (lbs. per sq. in., for example) in these two positions at any instant of time, will be denoted by p_m and p_n, respectively; while p_a will represent atmospheric pressure. The time which has elapsed up to any instant, from some convenient point of reckoning, will be denoted by t. By du we understand the change in the velocity u, occurring in a small time dt.

Taking the acceleration, $\frac{du}{dt}$, of the velocity u, as positive, in deriving general formulas, let us consider the forces which at any instant act to produce this acceleration in the horizontal prism of water between sections n and m. The mass is being accelerated in a horizontal direction. At the head of the prism, m, is the resisting pressure, $F_o p_m$; at the rear, n, the forward pressure, $F_o p_n$; both of these being horizontal. On the sides of the prism are the normal pressures, which do not affect the horizontal motion; and the fluid friction which is horizontal. This latter, according to Mr. Froude's deductions (skin friction), may be written equal to the product of the following four quantities: a co-efficient f (abstract number, about 0.006); the area of the rubbing surface (in this case the convex surface, πdl, of the prism of water); the specific volume of the liquid (here $= \gamma$, $= 62.4$ lbs. per cubic foot); and the height due to the velocity u, *i. e.*, $\frac{u^2}{2g}$. The mass of the prism of water is $\frac{Fl\gamma}{g}$, while its weight has no horizontal component.

Hence, writing accelerating force = mass × acceleration, we have,

$$F_o p_n - F_o p_m - f(\pi d)\, l\gamma \cdot \frac{u^2}{2g} = \frac{F_o l\gamma}{g} \cdot \frac{du}{dt} \qquad (1)$$

i. e., after reduction,

$$\frac{p_n}{\gamma} - \frac{p_m}{\gamma} = \frac{4fl}{d} \cdot \frac{u^2}{2g} + \frac{l}{g} \cdot \frac{du}{dt}. \qquad (2)$$

Now the distance from point n to a position k in the reservoir at the same level as n (but just far enough away to enable us to regard the water there as under simple hydrostatic pressure and without appreciable velocity) is so short, that the flow from k to n *will be considered as steady* at all times. A similar remark can be made for the flow from position m to the contracted section of the jet in the outer air. Hence, from the ordinary relations for steady flow in Hydraulics (Bernoulli's Theorem), putting $\zeta_E \frac{u^2}{2g}$ as the loss of head at the entrance E, and $\zeta_1 \frac{v_1^2}{2g}$ as that at E_1, (v_1 being the velocity in the short pipe, so that, from $F_1 v_1 = F_0 u$, we can write $v_1 = \frac{u}{x_1}$), we have

$$\frac{p_a}{\gamma} + h = \frac{u^2}{2g} + \zeta_E \frac{u^2}{2g}; \qquad (3)$$

$$\text{and} \quad \frac{p_m}{\gamma} - \frac{p_a}{\gamma} = \frac{v^2}{2g} - \frac{u^2}{2g} + \zeta_1 \frac{v_1^2}{2g}; \qquad (4)$$

$$\text{whence} \quad \frac{p_m}{\gamma} - \frac{p_a}{\gamma} = \frac{u^2}{2g}\left[\frac{1}{x^2} - 1 + \frac{\zeta_1}{x_1^2}\right]. \qquad (5)$$

Substituting from (3) in (2), we have also

$$\frac{p_m - p_a}{\gamma} = h - \frac{l}{g} \cdot \frac{du}{dt} - \frac{u^2}{2g}\left[1 + \zeta_E + \frac{4fl}{d}\right]. \qquad (6)$$

Various problems can be treated by the aid of these two equations, (5) and (6). As one of the simpler cases, let us suppose that at first a steady flow is proceeding in the pipe with velocity u_0, with the nozzle fully open ; and that instantaneously the valve is partially closed so that the ratio $F : F_0$, or x, is suddenly changed and remains constant at this new value. Under these circumstances, reckoning the time, t, from the instant when the valve was changed, we note that the differential equation, (6), deals with three variables, p_m, u, and t; but that by substitution of the value of p_m from (5) the number is reduced to two, viz.: u and t. In this final form of (6) we find it a simple matter to separate the variables and integrate, thus obtaining u as a function of t, and finally both p_m and v as functions of t. In other words, we thus secure the means of computing the value of the bursting pressure, $p_m - p_a$, at the part m of the pipe, for any assigned instant of time after the change of the valve : also the velocity v of the water in the jet, the volume of water flowing per second, $= Fv$, and the corresponding "kinetic power" (kinetic energy per second) of the jet ; at any instant (all of these being variable).

Again, if the nozzle had been originally shut and the water at rest, the effect of a sudden and entire opening of the nozzle, (that is of the assumption by x of a value x_1), can be traced, and the time determined necessary for the velocity in the pipe to assume any assigned value (less, of course, than the velocity of final steady flow). Here, again,

x is constant and the same method of elimination and integration can be followed as before ; but the initial conditions are different.

NOTE.—A peculiar problem is presented by supposing that during the progress of steady flow with open nozzle, the latter is instantaneously and *entirely* closed, (the pipe remaining rigid and rigidly fixed in place). This would give x constant and $= 0$; and consequently from eq. (5), u not being zero, we obtain $p_m =$ *infinity*. In an extreme case like this, however, the compressibility and elasticity of the water would be brought into play, and the pressure p_m would be obtained from the formula $p_m = \frac{u_0 E}{V}$; where u_0 is the original velocity of steady flow in the pipe, V the *velocity of sound in water*, and E the modulus of elasticity (of volume) of water. That is, we may look upon this as a case of longitudinal impact of an elastic prism, treated by St. Venant on p. 480*d* of his translation of Clebsch on the "Elasticity of Solid Bodies" ; where the method used is stated to have been first proposed by Babinet, in 1829. A somewhat similar method is used by Mr. J. P. Frizell (see Proc. of Am. Soc. Civ. Engineers for August, 1897) when treating this case of the instantaneous closing of a valve at the discharging end of a pipe ; with the added refinement of a consideration of the distension of the pipe itself under the great bursting pressure induced.

In the remainder of the present paper, the time of closing the valve is not supposed to be so short, nor the length of the pipe so great, as to bring into play the compressibility of the water.

Since in important situations valves are rarely so constructed as to admit of very sudden movement through even a portion of their range, we have now to deal with the gradual and continuous progress of the valve in closing, and the proper treatment of eqs. (5) and (6) (x being now a variable) for tracing the fluctuation of pressure thus induced at the section m of the pipe near the valve ; see Fig. 1.

In the article in the Journal of the Franklin Institute already referred to, the writer made trial of various assumptions, each expressing the variable x as a simple function of the time, t, reckoned from the instant of first movement of the valve in closing, such a relation being called a "law of closing". Thus, t_0 being the whole time occupied by the valve in closing, the "law of closing" first tried was $x = \frac{F}{F_0}, = \frac{t_0 - t}{t_0}$; that is, the portion of the opening of nozzle still unclosed at any instant was made proportional to the time still remaining, $(t_0 - t)$, (the "remaining time") before final closure. In another assumption x was made proportional to the square of the "remaining time". It was found that under these circumstances (making an assumption as to the "law of closing") integration of the differential equation [eq. (6)] could only be accomplished when h was put $= 0$. With a zero h, some numerical illustrations were worked out, and the maximum pressure induced during the closing was computed in each case, for an assumed initial velocity, u_0, of the water in the pipe. Later in the article hints were given as to a possible graphic integration of eq. (6) for the case of h not zero, the subject being then discontinued for lack of time.

Coming now to Mr. Latting's graduating thesis; in the earlier part of his work the treatment of the case where $h = 0$ was extended, first by making x proportional to the cube of the "remaining time", and then to the fourth power; with numerical illustrations showing the amount and position (in time) of the maximum pressure in each case. It was found that the higher the power the earlier the occurrence of the maximum pressure; t remaining the same; but that the smallest maximum pressure occurred for x proportional to about the second power of the "remaining time". (It should be added that in all these cases where h was zero the area F_1 was taken $= F_0$; in other words there was no nozzle; also friction was neglected.)

The next case treated (with h still zero) was where, after an instantaneous motion of the valve to a new position, the law of further gradual closing was sought such that the pressure at m should remain constant. This was the first case in which Mr. Latting abandoned the plan of assuming a law of closing, (*i.e.*, of assuming x as function of t), adopting instead the happy expedient of assuming a relation between such variables as to make simple or unnecessary, the integration of any differential equation. The law of closing corresponding to the initial assumptions was determined later.

This new plan was then applied to the case in Fig. 1 (h not zero, but with $F_1 = F_0$; *i.e.*, no nozzle) the following assumptions being made in turn:

First, the retardation of the velocity, u, in the pipe was taken as constant, *i.e.*, $\frac{du}{dt} = -\frac{u_0}{t_0}$; or $u = u_0\left[\frac{t_0 - t}{t_0}\right]$; in other words, the decreasing velocity in the pipe was to be directly proportional to the "remaining time". These two relations being introduced into eq. (6), we have immediately $p_m - p_a$ as a function of t, ($p_m - p_a$ being the bursting pressure at m) and hence from (5) we can determine what function of the time x must be, to produce the assumed law of retardation for u.

Similarly, in another case, u was made proportional to the square of the remaining time; in another, to the cube; in another, to the fourth power. A numerical example was worked out for each, the results tabulated and also exhibited graphically by curves, the whole time of closing being divided into ten equal intervals. As to the laws of closing thus obtained, it is noticeable in all of them that it is necessary for the valve at the beginning of the closing to instantaneously reduce the sectional area of the jet by a very large fraction (six to eight tenths) of its original value.

For the last case treated Mr. Latting took the actual data of the pipe with which, through the kindness of the Ithaca Water Co., he was enabled to perform experiments. This was an eight-inch pipe, 2395 ft. in length, placed in position to conduct water from the pumping station on Six Mile creek to a stand-pipe on the adjoining hill, at an elevation of some 300 ft. above the station. The "blow-off" branch of this pipe was fitted with a cap into which was tapped a short length of two-inch pipe, with valve-gate, to serve as nozzle. A steam gauge, previously tested, was inserted in the side of the pipe at the point m (Fig. 1). Co-efficients of resistance for four 90° and two 150° elbows were estimated and introduced into their proper places in the derivation of eq. (6). Co-efficients relating to the valve-gate and short pipe were obtained previously by direct experiment in the Hydraulic Laboratory of the College of Civil Engineering.

The data were as follows: $l = 2395$ ft.; $d = \frac{2}{3}$ ft.; $h = 302$ ft.; velocity of steady flow when the valve was fully open, $u_0 = 6.4$ ft. per sec.; $\zeta_1 = 0.37$; $f = 0.0056$; and the whole time of closing, as proposed, $t_0 = 25$ seconds. Also, $x_1 = 0.0586, = F_1 : F_0$.

The assumption of *uniform* retardation for the velocity, u, in the pipe, was made, *i. e.*, that

$$\frac{du}{dt} = -\frac{u_0}{t_0}; \quad \text{and} \quad u = \frac{u_0(t_0 - t)}{t_0}.$$

From eqs. (6) and (5), by making t successively equal to 0 sec.; 2.5 sec.; 5 sec.; 7.5 sec.; etc., up to 25 sec.; simultaneous values of $p_m - p_a$ and x were obtained. From these values curves have been plotted in the accompanying diagram, Fig. 2; the time, t, being taken as abscissa, (increasing from left to right).

The uniform decrease of the velocity u is shown by the straight line UB, whose ordinates diminish from 6.4 ft. per sec. at the beginning to zero at the end of the 25 seconds. The pressure at m (above the atmosphere) increases from 115.6 lbs. per sq. inch at the beginning to its maximum of 138.8 at the end of the closing. (Note that the axis from which pressure ordinates are laid off is considerably (80 lbs.) below OB.) As to the necessary motion of the valve-gate the value of x (or ratio of F the sectional area of the jet to that, F_0, of the pipe) have been plotted, giving as a result the curve XB. At the beginning x must change instantaneously from $x = 0.0586$ ($= RO$) to 0.0568 ($= XO$), [in order to make the pressure at m change suddenly from 107 lbs. (for steady flow) to 115.6], varying thereafter gradually, according to the ordinates of XB, which, though nearly straight toward the end of the closing, is a curve of the fourth degree. The actual position of the valve-gate itself, corresponding to a given value of x, must be found by taking into account the shape of the orifice under the edge of the gate and the contraction of the jet. By preliminary experiments in the Hydraulic Laboratory, Mr. Latting determined such relations in this respect as were needed for the proper manipulation of his apparatus.

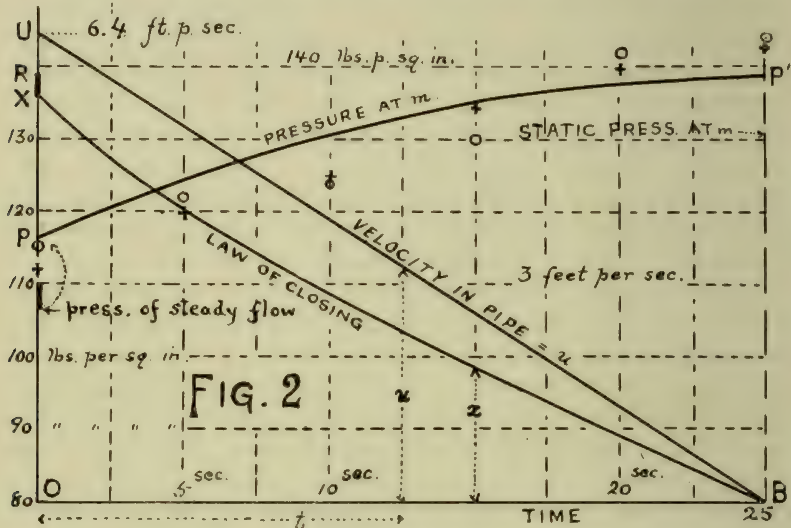


FIG. 2.

So far, only theoretical results have been referred to, in connection with Fig. 2. On account of the short time during which the pipe was available for his use, Mr. Latting made only two experiments as a test of the theoretical results just quoted. After a steady flow had been established, with the valve-gate fully open, further operations (to quote directly from the thesis) were as follows :

"The motion of the gate is regulated by plotting a curve the horizontal co-ordinate of which is time (in seconds), and the vertical co-ordinate the actual height of gate bottom from lower side of aperture. A slide moving vertically carries a pointer and is attached to the valve by means of a cord. The distance of this pointer from the horizontal axis on the paper gives the distance of the gate from its lowest position ; and, as the gate is lowered, the pointer, of course, moves down the page. An assistant, holding a watch, moved the paper on which the curve was plotted in a horizontal direction. The paper was moved just as fast as time increased, to the scale on which the abscissa, time, was plotted; and at the same time the operator moved the gate down so as to make the pointer trace the curve previously plotted. Another assistant read the gauge at the end of each five seconds."

The actual pressures as shown by the gauge in these two experiments are plotted in Fig. 2, a cross being used for Exper. 1, and a circle for Exper. 2. (As before stated, it must be remembered that the axis from which these pressures are laid off is 80 lbs. below the line OB.) The figures from which these pressures in Fig. 2 have been plotted are as follows:

Time from beginning, seconds.	Theoretical value of $p_m - p_a$ in lbs. per sq. inch.	Actual value of $p_m - p_a$ in lbs. per sq. inch in Experiment No. 1.	Ditto in Experiment No. 2.
0.0	115.6	112	115
2.5	120.1		
5.0	124.0	120	122
7.5	127.4		
10.0	130.6	125	124
12.5	133.0		
15.0	135.3	135	130
17.5	136.7		
20.0	137.8	140	142
22.5	138.5		
25.0	138.8	143	144

Of these values Mr. Latting remarks: "The discrepancies between the computed and actual pressures are due to the inability of the operator to move the valve exactly according to the required law. But these figures show a fairly good agreement between theory and experiment."

Attention should be called to the fact that in Fig. 2 the actual pressures are at first lower than the theoretical, while toward the end of the closing they are somewhat higher. This is noticeable in both experiments. The explanation would seem to be that the instantaneous closing up of about four per cent. of the opening under the valve at the beginning of its motion, as demanded by theory, in order to check the velocity of the water in the pipe in accordance with the assumptions made, cannot strictly be brought about. Even with the best approximation to it, the valve would be tardy in reaching its proper positions in the first part of the operation. Hence the velocity in the pipe would not be checked so rapidly at first as called for by theory, and consequently toward the end of the closing a greater rate of retardation, and hence a greater pressure at m, would be produced than that theoretically due, if the prescribed total time of closing be realized.

(Some further theoretical developments by the present writer will now be set forth.)

Let us now investigate the forms of the law of closing (referring again to the complete data of Fig 1) in two cases where the law of retardation of the velocity u in the pipe is so chosen as to avoid the necessity of an initial instantaneous movement of the valve through a finite portion of its range. This means that the pressure at m is not to change suddenly at the beginning of the motion ; to secure which effect it is evident from eq. (6) that we must have $\frac{du}{dt} = 0$ for $t = 0$. In other words, the velocity-line (such as UB in Fig. 2) must have a horizontal tangent at its initial point U, and hence cannot be a single straight line. It must, of course, pass through B, at the end of the time, t_0; and must be a smooth curve throughout if there are to be no sudden changes of pressure at m and (correspondingly) no instantaneous motions of the valve gate over finite portions of its range. That is, the inclination $\frac{du}{dt}$ must not change suddenly at any point.

CASE I. As a complete illustration of this, let us assume in Fig. 1 the following numerical data : Let $l = 3220$ ft. ; $h = 300$ ft. ; $\zeta_E = 0.50$; $f = 0.006$; $d = 8$ in. $= \frac{2}{3}$ ft. With a two-inch pipe as nozzle, we have $x_1, = F_1 : F_0, = \frac{1}{16}$, so that $x_1^2 = \frac{1}{256}$. Let ζ_1 be taken as 0.40.

For these data, by well known formulas of hydraulics, using the value 32.2 for the acceleration of gravity (for foot and second) we find the velocity of steady flow in the pipe to be $u_0 = 6.37$ ft. per sec. with the nozzle fully open ; and let us inquire at what rate (*i.e.*, $x =$ what function of t?) the sectional area of the jet must be reduced to completely stop the flow in the short time of $t_0 = 10$ sec., assuming the relation $u_0 - u = \frac{u_0 t^2}{t_0^2}$ for the rate of retardation of the velocity, u, in the pipe. The curve UB of Fig. 3, represents this relation and is nothing more than a parabola passing through B with its vertex at U and axis vertical ; that is, its tangent line at U is horizontal, to satisfy the condition that for $t = 0$, $\frac{du}{dt}$ shall be zero.

By differentiation we find

$$\frac{du}{dt} = -\frac{2\,u_0\,t}{t_0^2};$$

and hence, with the foot, pound, and second as units, we have, after substitution in eqs. (5) and (6),

$$\frac{p_m - p_a}{\gamma} = 300 + 12.74\,t - 1.830\,u^2\,; \tag{7}$$

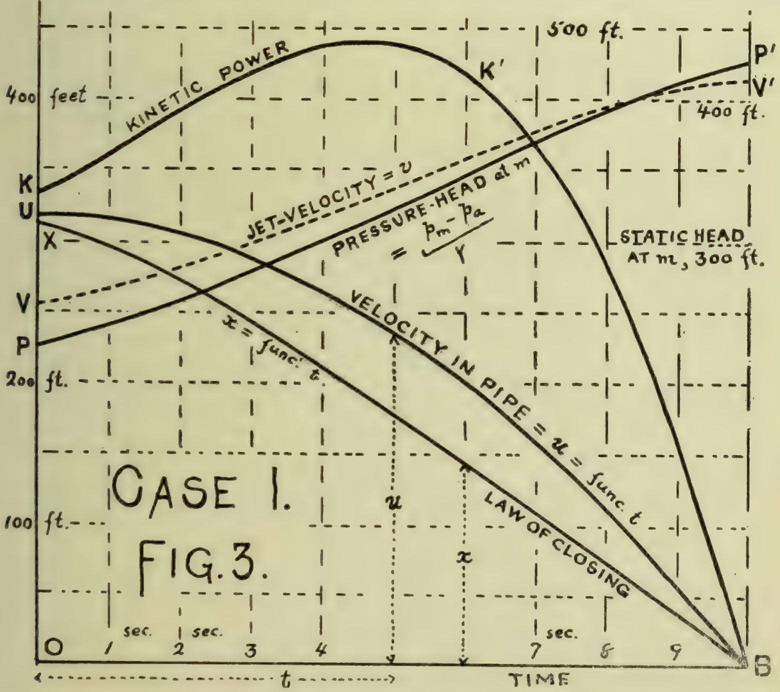
$$\frac{1}{x^2} = \frac{64.4}{u^2}\left[\frac{p_m - p_a}{\gamma}\right] - 101.4\,; \tag{8}$$

and, as already stated,

$$u = 6.37\left[1 - \frac{t^2}{100}\right]. \qquad (9)$$

The time t being successively put equal to 0; 1 sec.; 2 sec.; 3 sec.; etc.; up to 10 sec. (that is, the whole time of closing is divided into ten equal intervals); the writer computed the corresponding values of u, $p_m - p_a$, x, and v (velocity in jet), and plotted the corresponding curves shown in Fig. 3; in which the pressure-head $\frac{p_m - p_a}{\gamma}$ (feet) has been used instead of the pressure itself (bursting pressure at m, above the atmosphere).

In Fig. 3 we note that the curve XB ($x =$ func. of t) or curve of reduction of jet-area, is convex upward throughout; whereas in the case shown in Fig. 2, it is convex downward. The pressure-head at m in the present case increases steadily from 225.8 ft. at the beginning, to a maximum of 427 ft. at the very end of the closing, as we should expect since from eq. (6) it appears that the maximum pressure will occur at or near the time of the greatest value of $\frac{du}{dt}$; *i. e.*, where the tangent to the curve for velocity u makes its greatest angle with the horizontal (at B in the present case). The static head at m is 300 feet.


CASE I. FIG. 3.

The velocity, v, in the jet increases from 102 ft. per sec. (that of steady flow) to its maximum of 165.8 ft. per sec. at the end of the closing (at which time the sectional area of the jet is infinitely small). This is shown by the curve VV', Fig. 3.

Another quantity has also been computed and a curve plotted for it, viz.: the *kinetic power of the jet; i. e.*, the kinetic energy of the amount of water flowing per second in the jet (ft.-lbs. per second.) This quantity, K, is derived thus:

$$K = (\text{mass per sec.}) \times v^2 \div 2$$

$$= \frac{F_0 u \gamma}{g} \cdot \frac{v^2}{2} = \frac{F_0 \gamma}{2g} \cdot \frac{u^3}{x^2}; \tag{10}$$

and is represented by the ordinates of the curve $KK'B$. Its initial value (same as for steady flow) is 22,450 ft.-lbs. per sec. (some 40.8 horsepower), but it soon increases to a maximum of 29,900 ft.-lbs. per sec. at about the end of the first five seconds; diminishing therefrom to zero at the end of the whole ten seconds of closing. This illustrates the fact that if the closing of the valve is brought about by the governor of a jet-water-wheel, in order to diminish the power of the jet, the first effect of the operation is just the contrary of that intended.

CASE II. The effect of causing the maximum pressure to occur at about the *middle* of the time of closing will now be investigated by choosing a curve for velocity-reduction in the pipe, which, while as simple as possible, shall have its greatest obliquity at the middle of t_0, *all other data being the same as in Case I*, including the time of closing (ten sec.).

For this (compound) curve, parts of two parabolas have been selected, equal in dimensions, but one of them inverted, as shown by UTB in Fig. 4. At the mid-point T, they have a common tangent, forming together a reversed curve terminating in U and B and having a horizontal tangent (as required) at U; with no sudden change in $\frac{du}{dt}$. For the left-hand curve UT, ($t = 0$ to 5 sec.),

we have
$$u_0 - u = \frac{2 u_0 t^2}{t_0^2}; \tag{11}$$

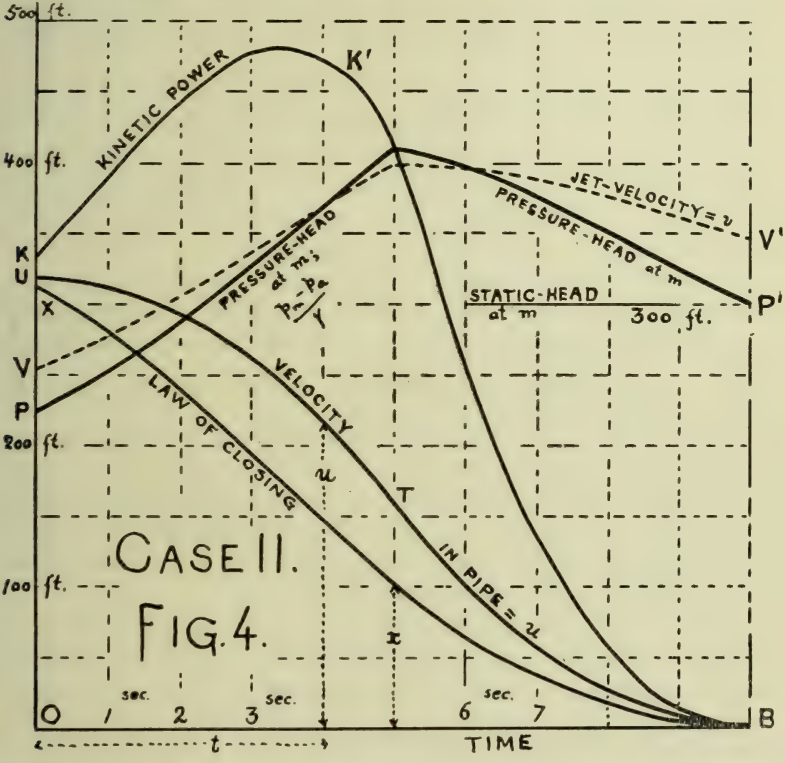
and
$$\frac{du}{dt} = -\frac{4 u_0 t}{t_0^2}; \tag{12}$$

while for $t = 5$ to 10 sec. (right-hand curve TB), we have

$$u = \frac{2 u_0 (t_0 - t)^2}{t_0^2}; \tag{13}$$

and
$$\frac{du}{dt} = -\frac{4u_0(t_0 - t)}{t_0^2}. \tag{14}$$

Assuming values of t differing by whole seconds from 0 to 10 sec., as before, and computing and plotting simultaneous values of the other variables, use being made of eqs. (5), (6), (11), (12), (13), (14), the curves of Fig. 4 have been obtained. In the curve for pressure-head at m, PP', we find that the maximum occurs at the middle of the time of closing and that its value is 409 ft., the initial and final being 225.7 ft., and 300 ft. (the static head), respectively. Therefore by this new assumption, the maximum pressure-head has been reduced from 427 to 409 ft. The curve for x, or "law of closing," proves to be a reversed curve, the first part of which with convexity upwards corresponds roughly to an increasing pressure; the latter part, with convexity downward, to a decreasing pressure. It will be noticed from this curve that the reduction of jet-area is proceeding most rapidly near the middle of the closing but is *quite gradual toward the end* (near B), differing markedly in this respect from the corresponding curve in Case I, (Fig.



3). The velocity, v, of the jet (see curve VV', Fig. 4) reaches its maximum value, 159 ft. per sec., at about the instant of maximum pressure; its initial value being the same, of course, as for the original steady flow, *i. e.*, 102 ft. per sec.; while the final value, 139 ft. per sec., is considerably less than that in Case I.

In the curve for Kinetic Power, $KK'B$, the maximum is reached at about 3.5 sec. from the beginning and has a value of some 32,000 ft.-lbs. per sec. This is a little greater than in Case I. The initial and final values are, as in Case I, 22,450, and zero, ft.-lbs. per sec., respectively.

From the two cases thus treated it is evident that for a fixed time of closing, t_0, the value of x should decrease rather rapidly at first, but very gradually toward the end (see XB, Fig. 4), in order to make the maximum pressure at m as small as possible; still, this should not be carried to an extreme, causing that maximum pressure to become greater, though occurring earlier in the motion.

As to the effect on the maximum pressure, of shortening the whole time of closing in the case of the single parabola for velocity u (Case I, Fig. 3, where the maximum pressure occurs at the end of the closing), we note by inspecting eq. (6), with $\frac{du}{dt} = -\frac{2u_0 t}{t_0}$, that the excess of the bursting pressure at m over the static pressure is inversely proportional to the whole time of closing.

This will also be true for Case II unless t_0 is quite large, in which case the maximum pressure may not occur at the middle, but near the end of the closing. For other assumptions of the relation $u =$ func. t, it would be necessary to investigate separately.

Another interesting and practical problem, whose details, however, will not be entered into here, is the search for the law of retardation (of velocity u in pipe) corresponding to a *constant* value of the pressure at m. A parabola like UT in Fig. 4, might be first assumed for a small fraction of the time of closing, so as to raise the pressure rapidly to the figure at which it is to remain constant during the remaining time. In this way probably the smallest maximum pressure could be obtained for a given time t_0; the corresponding proper valve motion being found later.

It is hoped that the foregoing theoretical discussions and the account of Mr. Latting's investigations and experiments will be interesting, and perhaps profitable, to those engaged in the design and management of long supply pipes conducting water to hydraulic motors of various types.

ORGANIZATION OF A RAILWAY ENGINEERING DEPARTMENT.

WILLARD BEAHAN, C.E., '78.

During an extended term of railway engineering work, equally divided in point of time between location, construction and maintenance of way, the writer always felt that on maintenance of way he did less real service for each dollar received than on location or construction. The late Wllliam Kerrigan, when General Superintendent of the Missouri Pacific Railway, asked the writer shortly after the latter had left maintenance of way on that system for work again on its new lines, if he did not wish to return to maintenance of way. The writer replied that he did not, for he felt that he did not earn, as the operating department was then organized, even the lower salary of maintenance work. To this fact Mr. Kerrigan gave ready assent. This experience is not unusual among railroad engineers who have seen considerable service in the three departments mentioned above. The fact should be borne in mind throughout this paper that, while the writer has a fair acquintance with many railroads, he has worked for but two,—the Lehigh Valley and the Missouri Pacific.

The object of this paper is to point out a better organization of the engineering department of our railroads with a view to saving money for the owners of the stock and bonds of the roads. All will concede that whatever can suggest economy of expenditures in American railways is now opportune. It is not the purpose of this paper to build up the engineering profession as such. Railroad corporations, like governments, are to be cared for, and not to care for employees. Railroad companies are largely, if not for the most part, made up of small stockholders who are frequently people of moderate wealth or widows or orphans. It is high time educated engineers awoke to that fact and shaped their walk and conversation accordingly. Every time a railroad passes into a receiver's hands it means destitution to many feeble people, or to children whose sole support was a dividend on that stock. Every reorganization means that another number of like incapacitated ones whose only dependence was in the bonds which were, of necessity, partially wiped out, shall suffer loss. A speech by the Hon. Chauncey M. Depew at Rochester last autumn is pertinent on this point. He stated, if my memory be correct, that the Vanderbilt family sold, as a matter of wise policy, some years ago, a large block of New York Central stock to an English syndicate. This sale removed from that family the ownership of a controlling interest. Mr. Depew stated the number of holders of stock, and called attention to the fact that the control of the road was in many hands. He showed clearly that his duties as president or the duties of a brakeman had a common aim. The president's duty to the stockholders was to make the road earn a dividend on the stock. Failure endangers dismissal to employee and entails suffering upon investor. It is seldom that one man or even one family owns the majority of the stock of a road. The reason is not far to seek, for experience has shown that owning a majority of the stock is not practically necessary in order to control the road. In voting, some stock always follows the strong holders. When, as engineers, we expect corporations to care for us, we are simply looking to our poor relations to support us. When we give less than our heartiest, manliest efforts to a railroad employing us, we are as culpable as the responsible employee of an asylum who sleeps at his post. This will be new doctrine to some. The sad part of it is that it is true and so growing in truth. Were it all false, it would still hold that whether one man or one hundred thousand men owned the road, our devotion is demanded just the same.

It seems necessary to explain to those not railroad men that the west has just emerged from the autocratic age in American railroading. Such phases have their uses, and must of necessity exist in the development of new countries. Hon. Cecil Rhodes apparently thinks the autocratic age has not yet passed in English colonization. Autocrats are necessary to young railroads, or to old ones in their aggressive periods. But, an autocrat is not an economist or, usually, a safe administrative officer. The late A. A. Talmage while with the Missouri Pacific Railway once sent an employee into a certain territory desired by that company for an important extension. His entire instructions to that employee were, "Go up and whip those —— —— fellows who are trying to get a line there." It was done. His line and not the enemy's occupied the desired territory. It cost some more to do it than it otherwise might, but the end justified the method. The economics of peace do not apply to war. But now there is surely profound peace in railroad construction. The late D. W. Washburn, then chief engineer of Mr. Gould's construction companies in Texas, once reprimanded a locating engineer. This engineer had been sent some four hundred miles to find a suitable crossing of a difficult river toward which preliminaries could be directed. He returned with an impracticable crossing, excusing himself by saying that the Indians were bad, his men complained about a shortage of water, etc. Mr. Washburn closed the subject in these words : "If you were dead no excuse would be necessary. As you seem to be alive, no excuse is sufficient." These laconic statements of two men of that past age exemplify my point. Surely "there were giants in those days." All honor to them, and all shame to those who criticise them now that they have passed away. Often, they died by reason of their own methods of management. Some of you will recall the newspaper headline, "You can't kill Tom. Potter," which appeared at the time Mr. Potter left the Burlington to take charge of the Union Pacific. You will remember that in two years he was dead. He was endeavoring to master every detail of the Union Pacific as he had mastered those of the road he had just left, but with which he had grown up. It always seemed to the writer that the climax of one-man power in American railways of the west was Mr. Talmage's death in his office car as he went about in almost ceaseless vigilance and study over his lines. Other incidents quite as pertinent will occur to the minds of my railroad readers.

This class of men taught the writer much in railroading, and tolerated not a few of his amateur blunders. Far be it from our wish to criticise departed friends. But the autocrat in railroading has left as a legacy a theory which, like the Jacksonian theory in politics, is most lamentable. That theory, *necessary to the autocratic system* of management, and therefore perfectly justifiable then, was that each subordinate must be the superior's "*man.*" Originally this meant that each subordinate must be the same type of man as his immediate superior, and with the same methods of work. That idea is necessary to the autocratic metbod of railroading ; and it is not the worst of railroad ideals, by any means. Carried to extremes, however, and especially when used by lieutenants and sergeants as well as by generals in railroading, it soon degenerates into saddling upon a corporation quite worthless men to do a little and do it poorly for a fair salary. Early in the writer's experience in railroad construction, the secretary of the company "hauled him up on the carpet," as railroaders say, for discharging the nephew of the president. The explanation was so unreserved and satisfactory that the writer was thanked instead of being discharged. Further, the secretary said that he would explain the matter to the president who wished to know if his nephew was lazy. Engineers too often retain worthless nephews in order, as they think, to retain their own positions. The above instance shows what the tenure of office of all relatives of all employees or officials should be.

There is another phase of this evil. A capable and well intentioned young man was once sent to report to the writer for duty on engineering work. This young man asked, *as a favor*, that the fact that his uncle was the vice-president of the road be kept from all his employees. "For," said he, "on the party where I have been they all knew who I was and when I did good work and got an earned promotion no one gave me any credit, but thought I got it because of my uncle." Because of our present lack of *esprit de corps* a young, capable engineer who is a relative of an official is soon shamed into leaving the road. There are many, no doubt, who have seen such cases. Few of this young man's fellow employees think for a moment that his promotion is on merit, for they know theirs is not, but is the result largely of personal friendship. The families who are large owners in railroads have no sons growing up in the engineering departments. They have in other departments, perhaps.

Even worse than relationship is the living of the "good fellow." A head of department once reprimanded the writer because his party was made up of men who would fight, the enemy at any time, and occasionally each other. The reply was, "I am so sick of nice '*good fellows*' that if you can show me a man in that old party who will not fight, I'll discharge him." A party of "*good fellows*" always has its cost of work per unit too high for economy. A man's *only* value on a party is measured by what he is capable of, and can be relied upon to do. May the writer be allowed to say here, with due modesty, that the secret of whatever success in field work has been his lot lies in that last sentence. There is absolutely no place whatever for any but *the best men obtainable*. These personal experiences in the personnel of engineer organizations seem more pertinent and brief than any abstract reasoning, hence their insertion. In closing this disheartening phase of our subject, it may be said, that during eighteen years' experience and observation in engineering parties, mainly in railroad work, the writer feels that two-thirds of all the men he has seen in subordinate positions below transitmen received and held their places through personal friendship rather than through merit. For obvious reasons, transitmen and engineers of all higher grades must be employed with at least some reasonable presumption of fitness. The evil does not, however, disappear in their case. Chiefs of parties and engineers of higher rank are amenable to the laws of business. When you wrong a stockholder to advance your own interests, nature's laws will punish you in your life's race.

Having fully considered the history and the growth of the evils, we pass to the remedies for them, hoping that the past may throw light upon the future. For example : What is the remedy for the evils of relationship and good-fellowship? The answer is quite obvious. Do away with hiring a man and appoint him instead. Use, in short, a form of civil service in appointment and promotion. The abuse came in through one-man power, now degenerated. Correct it where it started by having *one* man appoint, subject to the approval of *another* man.

The service rendered by maintenance of way engineers is too little. What is the remedy? It is the remedy Gen. Butler suggested when in '61 the negroes flocked to the Union lines, viz: "They are contraband of war. Put them at work on the fortifications." So, when a road has been built, it is the part of wisdom to put a sufficient number of the engineers known to be capable and reliable at work taking care of what they have built. The late George S. Noble, of the Texas and Pacific Railway, is said to have defined an engineering department as "a necessity during location and construction and a necessary evil in operating a railroad." If there be a department that is a necessary one, it is due to the stockholders that the manager make that necessary department a good and not an evil to the capital invested. Many railroad managers today have an engineer department merely because they have not succeeded in avoiding it, and think they are wise in having their engineers do an imperceptible service. They do not know how to make use of their engineers and most of the money paid for them is wasted.

No outline of railroad organization can have general application, of course. What is best suited for one road may be inefficient on all other roads. Nor should any organization be built around men and made to fit their peculiarities. This error is bringing pandemonium to excellent roads. Years ago a particular department had its limitations defined for a certain man. He filled the place, of course. He was eccentric in growth. Since his death no one has properly filled the place, or ever can. The place remains partially administered. All contiguous departments are still peculiar because this one man was peculiar. Again we say, corporations are here to be cared for and not to care for us.

In the broadest of generalizations, railroad management is divided under the following heads: Administration, Construction, Traffic, Transportation and Motive Power. With the department of Administration this paper is not concerned save to say that there is an officer of the department, usually called a Vice-President and General Manager, whom we shall term the General Manager. To this General Manager the heads of all other departments report, and through him their instructions are received. Over his head does no one report to some one beyond. This man is in some companies the President. The Traffic Department, at the head of which is the Traffic Manager, is now well defined on most railroads and with it this paper has naught to do. The Motive Power Department is also outside the province of this article.

The Construction and Transportation Departments are those with which civil engineers are associated. That they are two departments and not one has been the cause of much strife, many painful efforts at harmonizing, and too often an abandonment of it all with the consequent loss which failure entails. Let us consider where the difficulties of our problem lie.

Crankiness on the part of engineers and bullheadedness on the part of superintendents has caused most of the trouble. We have not fought *for* the company but fought the company. Whoever won, the stockholder lost. It seems to be conceded that one man cannot operate and another man maintain a division of a railroad. "But," says our engineer, who has more wit than pedigree, "I cannot be expected to work under a superintendent who was foreman on my track-laying gang in construction days or ran a material train but has now been promoted from roadmaster or trainmaster to superintendent." The right reply to this engineer is, "You will work for this company in whatever place its interests demand. Neither of us is the manager. It is our duty to obey and say nothing." Despite all efforts, railroads have not been able to maintain the track and bridges on a division without engineers. The office of General Road Master was created to do this by taking the track out of the hands of the superintendent when that superintendent was not a maintenance of way man. This type of road master has either passed away or is passing, we think. It was a wrong solution of a problem. The office of Superintendent of Bridges and Buildings has been a more lasting expedient, but this method is also passing away. This bridge department was instituted to take care of bridges on divisions where the superintendent of that division had been promoted from the train service or the track department. Again we say, it was a wrong solution. Each of these sub-departments has had separate engineer parties traveling over the same operating division. It has been sadly impressive to see three engineer parties file out of the same car each to do an hour's work and each wait half a day to start back, when any one party could have done the work of all. Each was given more mileage of the road, with a consequent added time of idleness on trains, usually working only one-third to one-tenth of their time each day. The men were being used on mileage, not on work. To have a

General Road Master in charge of the track of several divisions and a Superintendent of Bridges and Buildings in charge of structures over the entire road removes the maintenance of way supervision away from that division. The operating unit is the division. The maintenance unit is the division. Operation and maintenance belong together under one head on that division. Otherwise we have instructions by telegraph and inspection from rear platforms of specials, while the actual supervision is by some irresponsible person on the work. The Division Superintendent is the proper officer to have charge of operation and maintenance of way. His assistants must be men such as to allow of this Superintendents's being promoted from any department, that is, transportation, traffic, maintenance of way or motive power. He must succeed, if at all, through his executive qualities rather than through his special knowledge. As he is held responsible for maintenance of track, bridges and buildings he needs an engineer under him who has immediate charge of all maintenance on his division and only that. You cannot expect the superintendent to select him nor be responsible for his capacity or professional work. That is the duty of the Chief Engineer. But how can this be done, you ask, without conflict between the departments? It is done by having the right men for Chief Engineer and Manager.

Having carefully outlined the present difficulties of our question, we must now define and specify the parts of our organization. We are considering an organization for a fairly large system of the middle west. The principles and not the details are what we seek. Consider that the General Manager has powers as already indicated and that the Division Superintendents report to him. If there be General Superintendents intervening then they take the place in this paper of this General Manager, each for his portion of the road. This is a detail for a road. The Chief Engineer reports to the General Manager and has executive charge of location and constructive work and has no executive duties connected with track, bridges or buildings, except span-bridges and large buildings as hereafter stated. Unless this Chief Engineer be a locating engineer he needs under him a Locating Engineer who can take charge of the construction save in rush seasons. Necessity always rules the construction work and its organization is ordinarily un-economic. It does not now concern us save to say that a fit Chief Engineer needs no Superintendent of Construction. The Locating Engineer, we regret to say, now has a new field in revising bad location of our operated roads. Such work is a man-trap for a novice. The betterment of line and grade at span crossings is much needed as we erect new span bridges. The Locating

Engineer is the Surveyor at all times. Boundary disputes and questionable titles, and all that pertains to records of property acquired or line built is his province. As he should make the original records he must follow them up. He cannot do it all. He is not the office engineer. We re-survey too much now purely for our records.

The next man needed under the Chief Engineer is the Bridge Engineer, to take charge of the new construction or re-construction of truss bridges and important buildings. This sufficiently defines his work, which is more naturally described later.

We have thus far spoken only of the Chief Engineer and those who report to him. So far we have considered him as an executive officer. He must be in a measure administrative as well. We have said that while a Division Superintendent may not be a track or bridge man he should still be in charge of all maintenance of division. Obviously, there must be some one appointed to do this work for and under this Superintendent and for the capacity of this appointee others must be responsible. This Division Engineer, as we shall call him, could be appointed by the Chief Engineer subject to the approval of the General Manager, (or General Superintendent of that portion of the road). This Division Engineer shall have charge of the maintenance of track, bridges, buildings and water service on the operating division, with his office at the same point as the Superintendent, who is his immediate superior. His engineering assistants should be selected or removed subject to the approval of the Chief Engineer, and his engineering equipment furnished and maintained from that office. He should use the standard structures and keep his records in a standard way. A standard is designed to prevent repeated experiments at the expense of the same interests, and for uniformity. Nothing short of the approval of the General Manager and the Chief Engineer should make these standards valid. Reporting to the Division Engineer are the roadmasters who have the section foremen reporting directly to them. We will assume that this division has about five hundred miles of line. Under this Division Engineer is a General Foreman of Bridges and Buildings to whom all bridge foremen report directly. His office must be at the point which is headquarters for that division. He has charge of masonry and of water service. He does not have charge of renewals or repairs of truss bridges. Bridge inspectors report to the Division Engineer, and copies of inspections of truss bridges go thence to the Bridge Engineer, with the comments of the Division Engineer. The Bridge Engineer then becomes responsible for the safety of truss bridges, while the Division Engineer always does the policing of the division in all things. This Division Engineer is not supposed to be a bridge specialist but a bridge engineer in the sense that an engineer is who on the construction of a road has that title. Of the bridges other than span bridges he has full control under the Division Superintendent. Repairs or strengthening of truss bridges as decided upon by the Bridge Engineer may be carried by the Division Engineer or be done by contract under the approval of the Chief Engineer. Contract work on maintenance of way is seldom economical when company employees are competent. When a span bridge becomes too old or too light according to the Bridge Engineer, and the Chief Engineer with the approval of the General Manager decides to buy a new span, it is the duty of the Bridge Engineer to make the plans, advertise for bids, award the contracts, (subject to the approval of the Chief Engineer), check the strain sheets, attend to shop inspection, receive the bridge f. o. b. cars on the company's tracks, put in the foundations, erect the span, test it and turn it over to the transportation department. We have said erect the span, for a railroad company having a capable Bridge Engineer can save money by so doing. An iron erection foreman with a small outlay for tools and a half dozen iron-handling men is all that is necessary. This foreman and gang can be selected so as to be able to work on wooden bridges whenever necessary. Of course this Bridge Engineer must be one experienced in foundations below as well as above water. Pneumatic caissons and dredging cylinders need so much plant and demand a kind of labor such that the work would better never be attempted save by contractors who make such work a specialty. But coffer dam work has often to be done and requires only such plant as any railroad company owns. It is unpleasant for contractors to do pneumatic work for a railroad which has no bridge engineer familiar with that work, and it is uneconomical for the company not to have a man to attend to it intelligently. It is not difficult today to secure as a bridge engineer a man quite familiar with pneumatic work through his service with contractors, or some specialist in heavy bridges. Railroads waste much now in working at coffer dams. The writer knows this for he has done his share of it for railroads. In general, if the railroad companies realized what the inexperience and crankiness of their bridge department cost them there would be a revolution. In bidding on work for one railroad, contractors add an "arbitrary" to their usual price per pound just to cover the cost of the annoyances of that company's bridge engineer. This man the writer has known for years to be a most conscientious strain-sheet man. Yet he has no conception of bridge design, shop work or erection. There is a large class of engineers who believe that the more they make work cost a contractor the more that work is worth to the company. There are managers who are foolish enough to believe these engineers. There are stocdholders who pay for this folly.

We have said that the organization must fit the road and the engineer fit the organization. We shall now consider the kind of men these various engineers must be. We have intimated that the Bridge Engineer must be one who did not stop growing when he had learned to compute; that he must be a shop man; and an erection man. He must be a better and more experienced man than the Division Engineer. He should be a railroad man, and he must not be a crank specialist.

The Division Engineers must be men who have served an apprenticeship as instrument men on maintenance of way work. They cannot fill their positions with less than three years' experience in responsible work on maintenance of way. They must also have had experience in railroad construction, and in the handling of men. Experience and size count, youth is at a discount. They must understand every engineering principle they need to use, and not be men who are copying what they have seen others do. We cannot advance with our instruments always turned on the back sight. They must be men who have been taught the necessity in railroading of obeying orders without talking back. They must not be "scrappers." The qualities and discipline an engineer on railroads needs are those an army officer must have.

A Locating Engineer is a man for whom the railroad historian will have the deepest sympathy. He is the pioneer. He freezes, starves, thirsts, and develops those unlovely traits which isolation breeds. His health is usually broken by some chronic disease by the time he reaches his full mental growth. He sees his poorest, cast-off transitman outstrip him by glibness of tongue while he has been made dumb by habit and by having only a saddle horse to talk to. The newest roadmaster looks down upon him with scorn, and the new office boy of the Chief Engineer knows him not. Isolation on location work has warped some of our very best railroad engineers. What wonder that they are often eccentric! You will notice that in the organization outlined the Locating Engineer has only to deal with the Chief Engineer. Out of personal experience as well as observation was this suggestion made. What Chief Engineer who feels his responsibilities ever undervalued a good locating engineer? What other railroad man ever did aught else than under-value him. He has endured more exposure than any engineer will be called upon again to do in America. His devotion to his employers has been of a kind striplings know not of. His mistakes in judgment have been something terrible; they might have been more numerous. Without literature on his subject to aid him, with no precedents save those he had himself made, isolated, exposed, half exhausted, he has endured and grown warped. He needs to change. But inexperienced youngsters can never fill his place on our western lines. School men would be all at sea in his place and sink neath the wave of the first boom in new work. Nor can we afford to throw away the large sums of money the practice and experience of these old locating engineers have cost our railroads, just to let young men start in that same costly school of experience. As no former space was given the Locating Engineer in this article the writer has paused to pay him tribute in passing, and to outline the Locating Engineer as a character. Many of our profession have never formed his acquaintance. He will be even rarer in the future than in the past.

"I suppose," said the late D. W. Washburn, once, in Texas, "that I am in my present position because Mr. Gould's interests have considered me the best engineer here for Chief Engineer. *It is one thing to be the best engineer in the employ of a company, and it is another thing to be the best man for Chief Engineer of that company.* I have several men under me who in their line are better engineers than I am. I wish I had more such engineers under me." The writer has always felt that these were wise words and defined a railroad Chief Engineer's qualities. The first duty of a Chief Engineer is to work for the interests of the owners of the road, first, last, and all the time, and inspire in his subordinates the same spirit. The instant he considers self above company he is unworthy the position. He must be a judge of engineers and all their work. He must be a man under whom other engineers will do their best work. He must realize that intolerance is rampant in his profession among its older members. He must be a peacemaker and an organizer. He must be a man whom to meet inspires in the public a confidence in the company. He is the forerunner in a new country and must secure the good will of the people by the time that line is ready to operate. His integrity must be above suspicion, and his word as good to a contractor as the company's bond. He must be a construction man. He may be a location man. He must become a maintenance of way man, and ought to be one at the start. There is time to learn more maintenance of way. There is never time for him to learn construction. That work is rush work and detail work, and no man can, as Chief Engineer, build his first line economically or speedily. He must be a happy combination of railroad man, civil engineer, and business man, but first a railroad man for *his* road which he delights to serve and whose service creates for it friends among strangers, confidence among owners, and devotion among young engineering assistants. He must be like the lamented George B. Roberts, President of the Pennsylvania Railroad, of whom the press has just said: "Under his management the service became a clear civil service, and it was his effort to make even the lowest employee have a feeling of pride in the excellence of the great system."

The writer asks the opinion of the railroad members of the Association, especially, on his views and the principles here expressed. They are his views, not of today only, nor yet of yesterday. They belong to all days that have passed since he began railroading. They concern the owners of the vast and complicated railroads of America.

NOTES ON INSPECTION OF TIMBER STRUCTURES.

JUSTIN BURNS, C.E. '92.

Timber Railroad Structures with which young engineers will have the most experience are Howe truss bridges and trestles with pile or frame bents. Their first acquaintance with these will probably be in the capacity of an inspector in the engineering department of railroads where timber is of a sufficiently low cost to permit the building and rebuilding of wooden structures. To determine when a wooden bridge has outlived its usefulness and just how "ripe" it may be allowed to become are matters which require the judgment trained by experience. A year or two of service added to the life of a bridge or trestle means the saving of the interest of a year or two on the cost of renewal of the structures. For example take a division of 150 miles of track and say there will be 8,000 linear feet of pile trestle costing about $7.00 per linear foot. The cost of renewal would be $56,000. If by efficient inspection, the length of service is extended one year then with money at 5 per cent. there is a saving of $2,800.00 which would nearly pay the expenses of the Division Engineer and his assistants for a year. The argument in opposition to prolonging the life of worn out structures brings in the probability of a failure which would cause a wreck costing many thousands of dollars and instantly wiping out several years' savings in the economical bridge department. But proper inspection will reduce to a minimum the probabilities of a bridge failure, and a few notes are here given to those who may find it part of their duties to act as bridge inspector in the maintenance department.

The location of bad places in trestle work can often be discovered by merely observing the alignment of the rails and the surface of the track over the bridge. In fact, if the alignment and surface are found in good condition it is almost proof positive that the structure requires little further inspection. If the rails are found out of line at certain points it may mean a settlement of one side only of the bent or an entire shifting of that portion of the trestle.

If low places are discovered in the surface, it would indicate either the timber crushing or the bent settling ; in either case, it would be a place which required special inspection.

After noting these defective points, the ties and stringers are inspected. The ties should be in such condition as to hold the spikes firmly and prevent the rails from spreading. The stringers should be observed carefully for checks on the lower edge and for crushing upon the caps. Season checking in white pine stringers should be carefully distinguished from strain checks. Stringers and other large sized white pine timbers are generally cut to order in the mills and rarely have time to be properly seasoned to prevent deep checking before they are used in the construction. An excessively strained stringer will generally show an unusually large amount of season checks before it shows any failure from over-loading.

Then follows the inspection of each bent separately. The cap is examined for soundness, noticing the amount of crushing on the piles or posts. If the cap is dry rotted in the center it will crush and split over a small pile or tenanted post ; this is frequently found to be the case in the end bank bents where small piles are often used and where the rotting of the cap is facilitated by accumulations of roadway material carried over by the passage of the trains. The bearing of the cap on each pile is examined since defective construction or the settlement of one pile more than another will bring bending into the cap which is not considered in the design of caps.

The point at which the piles first show failure is usually where they enter the ground or water ; the alternate wetting and drying hasten the destruction of the fiber. The piles should be tested by boring with a small auger or bit and the manner in which the tool "takes hold" will indicate sound or rotten material. Old piles sometimes appear sound and seasoned on the outside but will be found to have only a ring of sound wood ; while others have merely a small portion of the heart wood intact and yet they act well under heavy loads. It will be found that the soils into which the piles are driven have different effects upon the life of the pile ; a fine loamy sand hastens the deterioration of the timber to a greater extent than a clayey soil.

The posts and sills of frame bents are examined for crushing and rotting caused by water working into the joints. The sills rest upon piles or "mud sills" and require an examination for crushing as they are most exposed to the action of alternate wetting and drying. If the foundation piles are below the water level so as to be constantly covered with water, they will need no investigation as the water preserves them from decay.

The sway bracing requires little attention ; if it is properly put on when the trestle is constructed, it requires no more inspection as it outlasts the other parts of the bents. The longitudinals are not used excepting in high trestles and they should be examined for crushing and longitudinal motion due to the thrust of the embankment. When a railroad crosses a deep valley in which there is a small stream which would not require a truss bridge, the immediate construction would probably be trestle work as the facilities for building the masonry and conveying the material for the embankment would be wanting. When this first trestle becomes fit for renewal, instead of rebuilding it, the fill can be easily made by trains hauling material from some convenient point upon the line. Often only the ends of the trestle will be filled, leaving some high trestle work in the middle to be rebuilt. The longitudinals in this work should be designed to resist the thrust of the settling embankment; and the inspection of the longitudinals should note the effect of this thrust upon the bents.

In regard to the inspection of Howe truss bridges, the condition of the alignment and the surface of the track will show any serious defect. The camber in the bridge can be observed by sighting along the upper or lower chord and the approximate amount noted. The passage of trains over the bridge should be utilized to observe the effect upon camber; each truss should be examined in this regard because often the camber is different and greater in one truss than in the other. This would cause a swaying in the train and it could be corrected by proper adjustment of the tension verticals.

If the Howe truss is a deck bridge, the track will probably be carried on ties resting upon the top chords of the trusses. These ties are usually 14″ or 16″ in depth, their size, of course, being dependent upon the distance between trusses. The ties are notched down a few inches over the chord to prevent their lateral movement; and in inspection, these notches should be observed for cracks or the splitting away of the lower portion of the tie. Theoretically the notched tie is as strong a beam as one of the same depth but unnotched; yet this square notch has a weakening effect as the timber tends to split off below the notch.

In a through Howe truss, the ties are carried similarly to those in the trestle, that is, resting upon stringers, and their inspection is identical with what has been stated before. The floor beams are suspended from the panel points by hangers; the point of probable failure is direectly under the stringer bearing and the failure is usually a fracture on the lower edge.

The upper chord needs little attention; being in compression, it only weakens by excessive rotting and by defective lateral bracing. The alignment of the upper chord would immediately display any serious defect.

The lower chord causes the greatest trouble and it is usually on account of its failure that the bridge is condemned. It is built of several pieces of timber bolted and keyed together, provided with splicing blocks at the joints. The splicing blocks are either of iron or oak and they are notched into the timber a certain distance back from the joints. To transfer the tension through the splicing blocks, the strength of the joint depends mainly upon the resistance of the timber to shearing along the grain between the notch and the joint. The first indications of overloading or failure in the lower chord will be a slight revolving of the packing blocks, the shearing at the splices and a longitudional movement of the angle blocks near the ends of the truss. All splices should be inspected carefully for any cracks or signs of pulling apart and especially to note any recent change.

In the web system, the verticals are inspected for equal distribution of stress; each rod should carry its proportional amount of the load and the difference in adjustment of the rods can be found by deflecting each rod in a panel point set, noting the relative ease or difficulty with which they are moved. The nuts should be marked and watched for any movement; the ends of the rods should project beyond the nuts so as to give full bearing to the threads.

The diagonals fail by crushing at the ends; moisture works into the ends of the posts over the angle blocks and weakens the fiber. Although in the design of the diagonals, the controlling requirement is the proper strength of the timber acting as a long column yet they fail by compression at the ends, thus acting like short blocks. The bearing of the diagonals is noted, especially that of the counter diagonals as they loosen up when the truss begins to "stretch out" or weaken; this loosening has a rocking effect when the wheel loads bring the counter diagonals into action.

The lateral systems are similar to the web system of the trusses and that inspection is on similar lines. All braces and rods should have their proper bearing and tension.

These are in the main, the important points of inspection and they may serve as suggestions to the inexperienced as to what details require the most attention.

In connection with these notes on inspecting timber construction, it might be of interest to state some of the extreme stresses under which old timber bridges are occasionally found working. Through inefficient inspection or a too zealously economical management, structures continue to carry loads several times in excess of that for which they were designed. The increase in weight of rolling stock of the present over that used in the bridge designs of fifteen or twenty years ago has been so great as to far exceed the provision of safety of even a low unit working stress; and timber is found working in excess of laboratory test breaking strain and performing its functions through "pure force of habit." If this same piece of timber be taken out and placed in another structure with much less strain coming upon it, the timber will invariably fail. Some cases of overloading white pine floor beams and stringers have come under the notice of the writer and although they have been brought to public notice before, a repetition may be allowable.

There was on one of the western railroads in the yard of one of its principal stations a combination Howe truss bridge of three spans of 175 ft. each and 12 panels to a span. The bridge was used only for switching purposes and a few freight trains ; the rate of speed of trains over the bridge did not exceed ten miles per hour and was usually about five miles per hour. The structure had been condemned by the engineering department but on account of peculiar circumstances and conditions, the management required the maintenance of the structure.

The floor system consisted of 6″ × 8″ pine ties placed on eight 7″ × 16″ stringers, three under each rail and two jack stringers. The stringers were supported at each panel point by three 7″ × 18″ white pine floor beams. The distance between centers of trusses was 16 ft. At some of the butting joints, the ends of the stringers had rotted so that they conveyed their load to the outside floor beam only ; the continuous stringers at the same panel points were also considerably rotted so as to convey little load to the middle floor beam. This threw nearly all the strain on the outside beams ; and in one case (due to repairs) the stringers were all butting joints at the panel point and conveyed all their load to the outside floor beam. Under the loads to which the bridge was daily subjected, the maximum stress per square inch in the outer fiber of the floor beams was 4950 lbs. There were numerous failures of floor beams in the bridge and almost invariably a check would start on the lower side of the beam and under the rail, and after cracking vertically two or three inches, the timber would split horizontally. Yet there were some of the floor beams of rather poor quality and full of knots which, for more than a year, had sustained this stress of practically 5,000 lbs. per sq. in. The only evidence of overloading was the presence of more horizontal checks than season checking would cause.

Another instance of excessive loading was that of a 14 ft. span in a pile trestle whose deck was of 6″ x 8″ oak ties on six 7″ x 14″ white pine stringers, two under each rail and two jack stringers. In this case, one of the two track stringers under one rail was so badly rotted as to convey no load to the bent, thus leaving the other stringer to be the only support. The stress in the outer fiber of the stringer caused by the engine wheel loads was 2,940 pounds per square inch ; while under a

6,000 pounds capacity car, the stress was 3,220 pounds. The working stringer showed no signs of failure.

The frequency with which the load is applied seems to effect the durability of overloaded stringers. There was another case in which the trestles were designed and built according to the requirements of the loading at the time of construction ; the unit working stress used in the design for strain in the outer fiber of the white pine stringers was 750 pounds. Shortly after erection the railroad company purchased and used heavier locomotives, which produced a strain in the outer fiber of the stringers of 1,500 pounds per square inch. Traffic over that division was much greater than in the cases above stated ; and, after nine years' service, the stringers failed by cracking on the lower edge, although otherwise they were in good condition and would have lasted several years longer under the loads for which they were designed.

The accompanying views show the condition of some Howe truss diagonals which were removed from an overstrained bridge. The counter diagonals seem to have fared worse than the main diagonals, which was probably due to their fitting loosely and the effect of being put under compression with impact as the change of wheel loads brought the counters under strain. Fig. 1 shows a panel point with a counter diagonal in position. Fig. 2 shows ends of diagonals cut off during repairs. The view was taken with these ends sticking up in the snow to facilitate photographing. An examination will show the failure of the diagonals to have been that expected from short columns ; the slight protuberance seen on the ends are places where the diagonals fit over rod holes in the angle blocks and they indicate the amount of compression the brace underwent. The size of the timber was 6″ x 8″.

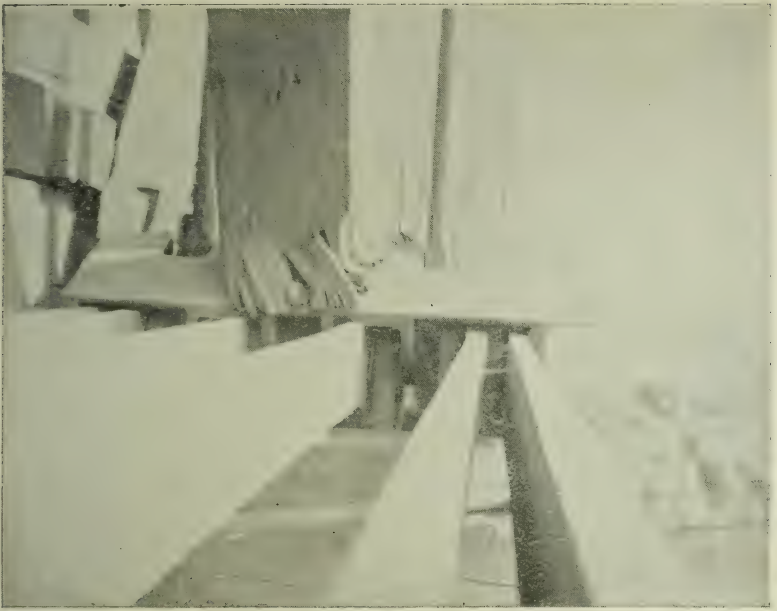


FIG. 1.

FIG. 2.

EXPERIMENTS AND STUDY UPON THE HOLDING POWER OF WOOD SCREWS.

PREPARED FROM A THESIS BY NORRIS M. WORKS, C. E., '97.

Only two series of tests of the holding power of screws, of any great value, were found by the author ; one made at the Watertown Arsenal on the "Holding Power of Nails, Spikes and Screws," Tests of Metals, 1884 ; the other by P. Lobben, originally published in the American Machinist, and afterwards in the Digest of Physical Tests, Riehlé, Phila., Vol. I, 1896.

In the first series, screws Nos. 12, 14, 16, and 18, and 1.5, 2, 2.5, 3 and 3.5 inches long, were tested in white pine perpendicular and parallel to the grain, also in yellow pine, white oak and California laurel ; three sizes of lag screws were also tested. In the second series lag screws were used in spruce, chestnut and pitch pine. Bevans also gives values for several small screws in beech, ash, oak, mahogany, elm and sycamore.

None of these tests take up a whole series of screws to bring out the effect of length and diameter upon holding strength. Only one series, P. Lobben's, shows anything in regard to the best diameter of hole to be used for the screws. The writer has attempted a complete series of tests with one kind of timber and one kind of screw to bring out as far as possible the effect of variations in diameter and length of screw and in size of hole.

All the tests were upon white pine in order to obtain a series of comparable results. Only three sticks, from different trees were used.

(A) was very clear and straight grained, 3 by 4 inches by 8 feet, specific gravity 0.42, rate of growth 9.5 rings per inch.

(B) was of the best quality of timber, thoroughly seasoned, 4 by 8 inches by 10 feet, with specific gravity of 0.47.

(C) was good clear stuff, seasoned at least four years, uniform in quality, fiber rather tougher than in either A or B, specific gravity 0.38, and rate of growth 15 rings per inch.

All the screws used in the tests were made and kindly furnished by the American Screw Co., of Providence, R. I.

TABLE OF SCREWS AS MADE BY THE AMERICAN SCREW CO.

No. of Screw.	LENGTHS.				DIAMETERS.					THREADS.						No. of per Pound.
	Total.	Shank	Thread	Head.	Shank.		Middle Thread.		Head.	Mean Depth	Number.	No.pr. Inch.	Effective Length.	Middle Circumference.	Area of Threads	
					Gage.	Actual	Outer.	Inner.								
4	1″	″.34	″.60	″.06	″.110	″.105	″.103	″.07	″.22	.016	14	22	″.47	″.323	″.15	482
8		.31		.09	.163	.158	.157	.10	.31	.028	8.5	15	.52	.492	.24	217
12		.30		.10	.216	.211	.208	.13	.42	.039	6.5	13	.41	.653	.27	117
16		.30		.10	.268	.268	.266	.18	.52	.045	6	13	.37	.836	.31	73
20		.23		.17	.321	.317	.310	.21	.63	.050	6	13	.39	.974	.38	49
4	1.5	.54	.90	.06	.110	.108	.107	.07	.21	.018	18	22	.62	.335	.21	329
8		.51		.09	.163	.159	.157	.10	.30	.028	12.5	15	.62	.492	.30	147
12		.50		.10	.216	.214	.214	.15	.42	.032	9.5	11	.65	.672	.44	82
16		.38	.95	.17	.265	.260	.260	.19	.53	.035	8	11	.69	.817	.56	52
20		.38		.17	.321	.320	.314	.20	.62	.057	6.5	8	.71	.986	.70	35
24		.30	1.00	.20	.374	.382	.355	.24	.75	.057	6.5	7	.70	1.11	.78	26
8	2	.66	1.25	.09	.163	.159	.156	.09	.32	.030	18.5	15	1.09	.49	.53	113
12		.64		.11	.216	.215	.212	.13	.43	.041	13.5	11	1.01	.66	.67	63
16		.60		.15	.268	.269	.263	.18	.53	.041	11	9	.95	.83	.78	40
20		.56		.19	.321	.318	.314	.20	.64	.047	9.2	8	.94	.98	.93	27
24		.55		.20	.374	.374	.367	.25	.72	.058	9	7.5	.96	1.15	1.10	20
8	2.5	.86	1.55	.09	.163	.162	.157	.10	.32	.028	22.5	15	1.25	.49	.61	91
12		.85		.10	.216	.212	.210	.13	.42	.040	16.5	11	1.34	.66	.88	51
16		.81		.14	.268	.262	.262	.18	.51	.040	14	9	1.35	.82	1.11	32
20		.78		.17	.321	.321	.316	.20	.62	.055	12	8	1.34	.99	1.33	22
24		.75		.20	.374	.369	.367	.26	.73	.053	10.5	7.5	1.30	1.15	1.49	16
12	3	.88	2.00	.12	.216	.213	.211	.13	.42	.038	21	11	1.54	.66	1.02	43
16		1.01	1.85	,14	.268	.265	.264	.18	.54	.042	16.5	9	1.59	.83	1.32	28
20		.95	1.85	.20	.321	.321	.317	.20	.63	.058	15	8	1.64	1.00	1.64	19
24		.89	1.90	.21	.374	.370	.367	.23	.72	.066	13.5	7	1.62	1.15	1.86	14

No. of Screw.	LENGTHS.				DIAMETERS.					THREADS.						No. of per Pound.
	Total.	Shank	Thread	Head.	Shank.		Middle Thread.		Head.	Mean Depth	Number.	No. pr. Inch.	Effective Length.	Middle Circumference.	Area of Threads	
					Gage.	Actual	Outer.	Inner.								
12	3″.5	1″.18	2″.20	″.12	″.216	″.216	″.208	″.13	″.42	.036	25	11	″1.93	″.65	″1.32	37
16		1.15		.15	.268	.264	.260	.17	.50	.045	20	9	1.96	.82	1.60	24
20		1.10		.20	.321	.315	.317	.22	.63	.048	17	8	1.83	1.00	1.83	16
24		1.08		.22	.374	.369	.362	.24	.70	.058	15	7	1.85	1.14	2.11	12
12	4	1.48	2.40	.12	.216	.211	.209	.140	.40	.034	25	11	1.95	.66	1.28	33
16		1.46	2.40	.14	.268	.268	.265	.180	.52	.042	21	9	2.04	.83	1.69	21
20		1.43	2.40	.17	.321	.321	.317	.210	.62	.053	19	8	2.00	1.00	2.01	15
24		1.28	2.50	.22	.374	.371	.369	.250	.70	.059	17.5	7	2.01	1.14	2.34	11
16	4.5	1.70	2.65	.15	.268	.265	.262	.180	.53	.041	23	9	2.25	.82	1.85	19
20		1.55	2.75	.20	.321	.315	.315	.210	.63	.052	22	8	2.34	.99	2.33	13
24		1.40	2.90	.20	.374	.375	.367	.250	.72	.058	19	7	2.39	1.15	2.75	10
20	5	1.70	3.10	.20	.321	.316	.315	.200	.62	.057	24	8	2.59	.99	2.56	12
24		1.40	3.35	.25	.374	.370	.363	.260	.75	.051	23	7	2.82	1.14	3.21	9
28		1.50	3.25	.25	.426	.424	.416	.275	.82	.070	18	6	2.47	1.32	3.26	7
20	6	2.12	3.70	.18	.321	.315	.313	.225	.63	.044	29	8	3.00	.98	2.95	10
24		2.10		.20	.374	.374	.370	.270	.75	.049	25.5	7	3.12	1.16	3.62	7
28		2.07		.23	.426	.417	.416	.310	.83	.053	22	6	3.08	1.31	4.03	6
30		2.05		.25	.452	.451	.451	.330	.87	.060	21.5	6	3.00	1.41	4.24	5

The above table was made up from the screws actually used. The "Effective length" of the threaded portion was found by subtracting from the total an allowance for the small diameter of the sharp tapered portion at the point. The "Middle circumference" is that for the outside of the thread at the center of the portion taken for the effective length. The "Area of threads" is the product of the effective length and the middle circumference; it is used to represent the shearing area in the wood, as assumed in the Watertown Arsenal Reports.

A few tests were made of the "Rogers drive screw," a screw having so steep a pitch that it will revolve in driving into wood with a hammer.

Most of the screws were inserted with a hand brace; a wrench with a thin piece of steel in the slot was used for the larger ones, while in some of the experiments the screw was driven, in whole or in part, with a hammer as noted.

When holes were bored, they were bored with a Morse twist drill to the depth of the portion of the thread having a uniform diameter, so that the point would enter the wood at the bottom of the hole. A small auger hole, about ⅛ inch deep, was bored in nearly every case, in the bottom of which the screw was inserted, to prevent the slivering of the surface when the screw was pulled. When bored holes were not used, the screw was started with a hammer to the depth of a thread or two, and then turned in with the screw-driver.

When the screws were lubricated it was done with vaseline, lard oil or tallow. The screws were pulled with a slow steady motion in a 50,000–lb. Olsen testing machine.

The following preliminary tests were made in order to determine the most favorable conditions for the maximum holding strength for the full series.

(a) Tests with different sized holes for the same sized screw.

(*a*) TESTS WITH DIFFERENT SIZED HOLES FOR THE SAME SIZED SCREW. SCREW PERPENDICULAR TO GRAIN, IN WHITE PINE, STICK B.

No. of Screw.	Length.	Diam. of hole.	Maximum pulling force in pounds.						
			1	2	3	4	5	6	Av.
12	4″	″.00	1280	1280	1210	1190	1230	1380	1261
		.138	1320	1200	1270	1230	1260		1256
		.149	1260	1190	1230				1216
		.161	1180	1170	1170				1173
		.171	1230	1140	1150				1173
		.182	1030	950	950				976
20	3.5	.00	1430	1460	1400	1350	1400	1430	1411
		.171	1470	1420	1535	1500	1475		1480
		.187	1390	1440	1550	1500			1470
		.198	1400	1550	1450	1450	1510	1470	1463
		.209	1380	1520	1410	1450	1650	1620	1505
		.219	1400	1430	1400	1500	1360		1418
		.226	1370	1480	1400				1416
30	6	.00	3740	3750	3540	3970	4090		3818
		.279	3600	3810	4020	3970			3850
		.296	3450	4160	3450	3400			3615
		.327	3340	3870	3300	3610	3700	3470	3548
		.340	3720	3400	3440				3520
		.359	3130	3400	3330				3287

SCREW PARALLEL WITH GRAIN, IN WHITE PINE, STICK B.

No. of Screw.	Length.	Diam. of hole.	Maximum pulling force in pounds.						
			1	2	3	4	5	6	Av.
12	4″	″.00	1220	1120	960	1060	830	730	
		.00	1000	590	970	1010	1060		959
		.138							1146
		.149							1040
		.161	1170	1020	920				1036
		.171	980	910	960				950
		.182	870	1000	1000				956
20	3.5	.00	1220	1480	1160				1286
		.171	1300	1450	1180				1310
		.187	1200	1200	1370				1256
		.198	1130	960	1150				1080
		.209	1320	1460	910				1230
		.219	1180	1260	1420				1286

An examination of the table shows that the hole for maximum average strength has a diameter of .138/.14=99% ; .171/.22=78% ; .279/.33=84% ; respectively, of that of the body of the screw, when perpendicular to the grain, and of .138/.14=99% ; .171/.22=78%; respectively, when paralel with the grain.

That is, the diameter of the hole should be from 80% to 99% of that of the body of the screw for maximum holding strength, the falling off in strength as the diameter varies either way from these limits being greater when the screw is parallel with the grain than when it is perpendicular.

(b) Tests of screws, lubricated and not lubricated.

White pine stick A was used for these tests with No. 20 screws, 4 inches long, inserted the length of the threads in holes 0.198 inch in diameter. The mean of 4 tests not lubricated gave 1992 pounds, and of 4 lubricated, 2092 pounds, an increase of 5%.

(c) Tests of screws driven part way with hammer.

WHITE PINE STICK A. SCREWS PERPENDICULAR TO THE GRAIN INSERTED TO THE HEAD. NO BORED HOLES.

No. of Screw.	Length.	Pulling force—Screw inserted with screw-driver.					Depth with hammer.	Pulling force—Screw driven with hammer to depth indicated, then with screw-driver.					Ratio Average Strengths.
		1	2	3	4	Ave.		1	2	3	4	Ave.	
8	2″.5	850	840	820		837	1″	610	605	670		628	0.75
12		1030	930	1000		987		840	780	800		807	.82
16		1060	1140	1060		1087		910	840	930		893	.82
20		1300	1400	1330		1343		1000	1060	1050		1037	.77
24		1450	1350	1450		1417		1050	1080	1150		1093	.77
12	3.5	1330	1290	1300		1307	1.75	1220	1190	1210		1207	.92
16		1360	1430	1420		1403		1370	1270	1390		1343	.95
20		1970	1640	1510	1680	1700		1490	1480	1650		1540	.91
24		1840	1870	1830		1847		1750	1790	1570	1500	1652	.89
12	4	1380	1350	1400		1377	2	1190	1280	1490	1280	1310	.95
16		1590	1600	1860	1700	1687		1470	1620	1770	1540	1600	.95
20		1700	1850	2200	1870	1905		2250	1690	1800	2030 2030	1960	1.03
24		2230	2200	2480		2303		1940	1850	2120	2160	2017	.88
12	3	1260	1390	1280		1310	1.5	930	970			950	.72
16		1250	1420	1460	1470	1400		1180	1130	1370	1070	1187	.85
20		1840	1980	1980		1933		1720	1420	1470		1572	.81

WHITE PINE STICK A. SCREWS PERPENDICULAR TO THE GRAIN. DRIVEN ONE-HALF THE THREADED LENGTH WITH HAMMER, THE REMAINDER OF THREADED LENGTH WITH SCREW-DRIVER.

No. of Screw.	Length.	Pulling force—Screw inserted with screw-driver to root of thread.	Pulling force—Screw driven one-half way with hammer, as indicated above.					Ratio Average Strengths.
			1	2	3	4	Ave.	
12	3″.5	1080	820	820	800	730	792	0.73
24		1514	1320	1150	1350		1273	.83
20	6	2608	2070	2130	2200		2133	.81

An examination of the table shows that when the screws are driven one-half the length of the thread in white pine with a hammer and the remaining distance to the head with the screw driver the loss in holding strength is from 5% to 25%. In this case the screw thread imbeds itself nearly the whole length in new wood, just as it would do if screwed in from the surface with no hole to receive it. The shank fills the portion of the hole made by the hammer, and as its holding strength is small as compared with the whole, the disturbance of the wood fibers around it by driving has but a small effect.

When the screws are driven one-half the depth of the thread with a hammer, and the other half with a screw driver, the loss as indicated is from 17% to 27% as compared with screws driven the length of the thread with a screw driver alone.

(d) Tests of screws driven to the head, with various sized holes bored for the entrance of the shank.

The screws were No. 20, 3.5 inches long. The threads were inserted in a hole 0.233 inch in diameter, and the shank one inch into holes of different diameters as given.

Diameter Shank hole.	PULLING FORCE IN POUNDS.					Pulling force, thread only.	Ratio total to thread only.
	1	2	3	4	Ave.		
″.233	1630	1600			1615	1527	1.06
.248	1690	1940	1760		1797		1.17
.310	1690	1460			1475		.97
.279	1530	1665	1700		1631		1.07
.265	1630	1820	1590	1670	1677		1.10

The pulling force for thread only in the same stick and with the same sized hole was found to be 1450, 1450, 1660, 1550; or 1527 pounds as the mean of four tests. This value is used in the seventh column above.

An examination of the table shows that the greatest strength is for the hole of diameter 0.248 inch, or .248/.321 equals 77% of the diameter of the shank. This is about the same per cent as found for the inner cylinder of the threaded portion.

A study of the following table will show that the holding power of a wood screw in white pine is nearly in proportion to the area of the thread (see p. 64) in contact with the wood. This value decreases slightly as the screw lengthens, or the diameter increases.

It is probable that this law of holding strength will apply to different kinds of timber nearly enough so that a number of tests with a single good sized screw would be sufficient to determine the values for any screw in that kind of wood, having given its dimensions.

The Rogers drive screw. Six tests were made by driving with a hammer No. 12 screws 2 inches long, the effective length being 1.10 inches, and the surface in wood 0.74 square inch, giving an average pulling force of 433 pounds. This was increased 15% by unscrewing

HOLDING STRENGTH OF SCREWS IN WHITE PINE (STICK C.)

Screws were all lubricated in driving except the smaller sizes, and were inserted only to the shank.

PERPENDICULAR TO THE GRAIN.												PARALLEL WITH THE GRAIN.									
No. of Screw.	Length.	Diameter of hole.	Depth of hole.	Effective length.	Surface in wood.	Pulling force in Pounds.						Diameter of hole.	Depth of hole.	Pulling force in Pounds.						Ratio parallel to perpen.
						1	2	3	4	Av.	Per sq. inch of surface.			1	2	3	4	Av.	Per sq. inch of surface.	
4	1″	0″.0	0″.0	0″.47	0″.15	200	190	180	200	194	1268	0″.0	0.0	100	140	100		113	738	.58
									200											
8				.52	.24	240	250	230	270	247	1021			210	170	150		176	728	.71
12				.41	.27	340	350	310	330	332	1240			300	270	270		280	1045	.84
16				.37	.31	420	430	430	440	430	1387			220	240	280	230	242	780	.56
20				.39	.38	470	410	390	400	417	1096	.171	.4	300	280	270		283	745	.68
4	1.5			.62	.21	300	300	320	280	300	1428	0.0	0.0	145	200	200	170	179	852	.60
8				.62	.30	400	380	410	360											
									380	386	1266	0.0		240	280	310		276	907	.72
12				.65	.44	595	500	600	580	569	1303	.138	.5	370	430	440		413	946	.73
16		.152	.5	.69	.56	600	630	650	530	602	1067	.149		450	400	450		433	769	.74
20		.198	.5	.71	.70	710	700	670	660	685	979	.171		440	500	500		480	687	.70
24		.226	.5	.70	.78	810	800	780	760	787	1008	.219		550	720	620		630	807	.80
8	2	.0	.0	1.09	.53	580	480	530	510	525	983	.0	.0	450	550	410		470	881	.89
12		.0	.0	1.01	.67	850	830	760	730	792	1183	.138	1.0	500	540	570		536	800	.67
16		.152	1.0	.95	.78	870	810	850	900	857	1098	.149		630	670	660		653	838	.76
20		.198	1.0	.94	.93	960	960	920	870	937	1012	.171		650	650	680		660	714	.70
24		.226	1.0	.96	1.10	1100	1120	1110	1080	1102	997	.219		850	860	720		810	731	.74
8	2.5	.00	.0	1.25	.61	770	790	730		763	1240	.00		500	470	430	440	460	747	.60
12		.138	1.25	1.34	.89	890	960	1010		953	1072	.138		740	890	590		740	832	.78
16		.152	1.25	1.35	1.11	1215	1220	1120		1185	1067	.149		770	740	880		796	717	.67
20		.198	1.25	1.34	1.33	1300	1260	1320		1293	974	.171		990	830	1110		976	734	.76
24		.226	1.25	1.30	1.49	1370	1420	1360		1383	929	.219		950	1060	1000	1170	1045	702	.75
12	3	.138	1.5	1.54	1.02	1240	1240	1300		1260	1235	.138		790	770	940		833	816	.66
16		.152	1.5	1.59	1.32	1290	1290	1270		1283	972	.149		930	1140	1150		1073	813	.84
20		.198	1.5	1.64	1.64	1440	1390	1380		1403	857	.171		1020	1020	980		1006	614	.71
24		.226	1.5	1.62	1.86	1630	1720	1670		1673	900	.219		1330	1380	1390		1366	734	.82

PERPENDICULAR TO THE GRAIN.												PARALLEL WITH THE GRAIN.								
No. of Screw.	Length.	Diameter of hole.	Depth of hole.	Effective length.	Surfaces in wood.	Pulling force in Pounds.						Diameter of hole.	Depth of hole.	Pulling force in Pounds.						Ratio parallel to perpen.
						1	2	3	4	Av.	Per sq. inch of surface.			1	2	3	4	Av.	Per sq. inch of surface.	
12	3″.5	″.138	1″.75	1″.93	1″.32	1510	1340	1200		1350	1024									
16		.152	1.75	1.96	1.60	1530	1500	1540		1523	952									
20		.198	1.75	1.83	1.83	1690	1700	1750		1713	937									
24		.226	1.75	1.85	2.11	1930	1900	1850		1893	898									
12	4	.138	2.25	1.95	1.28	1715	1630	1610		1650	1288	.138		1220	1080	870		1056	825	.64
16		.152	2.25	2.04	1.69	1790	1770	1800		1786	1057	.149		1400	1200	1330		1310	775	.73
20		.198	2.25	2.00	2.01	2220	2080	2000		2100	1044	.141		1670	1350	1580		1533	764	.73
24		.226	2.25	2.01	2.34	2170	2310	2260		2246	958	.219		1570	1710	1710		1663	712	.74
16	4.5	.165	2.9	2.25	1.85	1780	1700	1680		1720	930									
20		.198	2.9	2.34	2.33	2180	2010	2090	2010 2130	2084	894									
24		.226	2.9	2.39	2.75	2530	2330	2530		2473	900									
20	5	.198	2.9	2.59	2.56	2550	2480	2500		2510	980	.141		2180	2200	1520	1940	1960	766	.78
24		.226	2.9	2.82	3.21	3180	2890	2920		2996	935	.219		2470	2110 2680	2300 2000	2070 2290	2206	689	.74
28		.265	2.9	2.47	3.26	3080	2910	2810		2933	897	.248		2320	1810	2210 2420	2000 2000	2127	653	.73
20	6	.198	3.25	3.00	2.95	2840	2790	2610		2746	932	.171		2000	2170	2160	2220	2137	725	.78
24		.226	3.25	3.12	3.62	3190	3160	3280	3190 2960	3156	871	.219		2570	2240 1970	2610 2370	2240 2670	2381	657	.75
28		.265	3.25	3.08	4.03	3580	3730	3460	3840 3210	3564	884	.248		2230	2560	2560 2480	2560 2610	2488	617	.70
30		.279	3.25	3.00	4.24	3920	4250	3670	4180 3535	3911	922	.279		2750	2520	3150	2570 2740	2746	647	.70
		Mean values,									1062								764	.723

and reinserting with a screw-driver, and it was reduced 21% by lubrication with vaseline before driving. Similarly the pulling strength of a No. 14, 2 inches long, was found to be 392 pounds, and this was increased 13% by reinsertion with a screw-driver, as above, and diminished 16% by the use of vaseline in driving.

Methods of failure. To determine the method of failure of a wood screw, a number of large ones were partially pulled and the wood then split apart to show the condition of the fibers. When parallel with the grain, the screw failed by a simple shear of the wood. When perpendicular, the screw failed by the bending over of the fibers which projected between the threads, the disturbance often extending lengthwise of the fibers for quite a distance beyond the threads, the holding strength thus depending upon the bending strength of the fibers and the compressive and shearing strengths of the wood. In addition to this is the frictional resistance of the shank if not inserted in too large a hole.

Conclusions. 1. Wood screws, when drawn parallel to the grain, depend for their resistance upon the shearing strength of the wood parallel to the grain.

2. When pulled perpendicular to the grain, the strength is a function of several conditions: of the shearing strength of the wood; parallel to the grain; perpendicular to the grain, and across the grain in a plane parallel to the fibers; also of the flexural stiffness of the fibers.

3. The maximum holding strength when pulled perpendicular to the grain, is obtained with no hole bored to receive the screw or with a hole about 0.8 the diameter of the inside cylinder of the screw.

4. The maximum holding strength, when pulled parallel to the grain is obtained when the screws are inserted in a hole of about half the same diameter as above. There is a distinct loss in the holding power of the screw, when inserted parallel to the grain, if no hole is bored for it, or if the hole bored is too small.

5. The work required to insert a screw in wood with no hole or too small a hole is so great that it is doubtless the most economical method to insert all screws in holes, slightly smaller than the inside cylinder of the screw. Insertion without holes is very likely to split the timber.

The fibers of the wood are crushed and destroyed when the screw is inserted with no hole or with a small hole. It is probable that such a screw will lose its value in the course of time under a constant or repeated load.

6. The necessity for a hole in which to insert the screw increases

with the diameter of the screw. It becomes imperative in the case of lag screws.

7. The loss in holding power on account of a slight crack is very small.

8. The increase in holding power due to the insertion of the shank is very small. On the contrary, the splitting tendency induced is likely to weaken such power.

9. The holding strength of a screw is proportional to the area in contact with the wood.

10. There is a small increase in holding power if the screw be lubricated. There is a loss if drive screws be lubricated.

11. The holding strengths of screws per pound weight is greater for the smaller screws. It decreases with an increase of diameter, but remains practically constant for a constant diameter, for different lengths.

PLACING CONCRETE IN NINETY FEET OF WATER.

DANIEL W. MEAD, C. E., '84.

During 1897 the writer was under contract to build a plant for increasing the water supply of Rockford, Illinois. This contract included, among other things, a shaft fourteen feet in diameter and eighty feet in depth, from the bottom of which extended a smaller shaft about six feet in diameter which was excavated to an additional depth of twenty-five feet. The large shaft was sunk by the pneumatic process through fifty-five feet of water bearing sand and gravel into thirty-five feet of clay. It was thought that this clay was about one hundred feet in thickness and impervious to water, so that when the larger shaft was finished the air pressure was removed and the smaller shaft sunk in the open. At a total depth of about one hundred and five feet a flow of water was encountered, which rapidly increased in quantity and drove the men from the shaft, filling the same to within about fifteen feet of the top and to a total depth of about ninety feet.

It was thought that if the bottom could be closed by ten feet of concrete bearing against the iron sides of the small shaft that the work might be pumped out and still completed in the open. Although this was afterwards found not to be the case, yet the method of placing the concrete may not be without interest. A number of possible methods of placing a homogeneous concrete bed were considered. The method adopted was as follows:

The inrush of the water had brought a considerable quantity of fine sand into the shaft. To remove this a Nye vacuum pump was fitted with one hundred feet of suction pipe and the end of the pipe let down near the bottom of the shaft.

The actual lift was, of course, only the fifteen feet from the water surface to the top of the shaft, but the water being pumped came from the bottom of the shaft and brought with it the sand it was desired to remove.

The concrete was to be sent down through an eight inch wrought iron pipe, which was screwed together of such a length that its lower end rested on the bottom of the small shaft and its upper end was about eight feet below the top of the large shaft. The lower end was stopped with a wooden plug before it was lowered into place, and the upper end was provided with a hopper into which the concrete could be thrown. The pipe was also suspended from a derrick and could be raised or lowered as desired.

The end of the pipe was allowed to rest on the bottom until the pipe was filled with concrete. It was thought that when the filled pipe was raised the pressure of the concrete would force out the plug and the concrete would begin to flow from the pipe.

After filling the pipe and raising it from the bottom it was found, however, that such was not the fact, and the pipe had to be pulled up and uncoupled a length at a time and the concrete removed. This was easily accomplished, however, for as soon as each piece of pipe was unscrewed a slight jar on the pipe caused the concrete to slide out. When the last length, which held the plug, was reached it was found that the plug had evidently leaked and the lower length of pipe had filled with water. As the concrete was thrown in, this water had separated it into its elements. The stone and gravel were at the bottom, the sand next, and near the top of the pipe was a plug of neat cement which had plugged the pipe effectually and held the concrete above from pressing against the wooden plug and forcing it out.

A second plug was arranged so as to make the pipe practically water tight and the pipe was again placed in position This time a smaller pipe one and one-half inches in diameter was placed inside the large pipe to be used if necessary to knock out the wooden plug and work the concrete loose.

This time when the pipe was filled with concrete and raised from the bottom the wooden plug was easily forced from its place and the concrete began to descend. As fast as the concrete sank from the hopper more concrete was thrown in and the pipe was thus kept full at all times. When the concrete in the pipe settled too slowly the pipe was raised to give more bottom opening and thus hasten it. If it sank too fast the pipe was lowered and the opening thus throttled. By this means the stream of concrete was easily controlled.

It is probable that only the first of the stream of concrete was delivered into the water and that the balance was delivered below a layer of six to twenty-four inches of the concrete already deposited.

In this way a mass of concrete ten or twelve feet in thickness was deposited under from eighty to ninety feet of water. Only the upper six or eight inches showed any signs of washing, this was soft and never set. The lower portion of the mass as far as determined was solid and as homogenous as when shoveled into the hopper above.

IMPACT TESTS ON WIRE ROPE.

ABSTRACT OF THESIS BY CHARLES F. HAMILTON, C. E., '97.

In looking up the subject of the strength of wire rope the author found numerous tests of tensile strength under gradually applied loads, but almost nothing upon the strength under impact. On this account he decided to make a comparison between the tensile and impact strengths of wire rope.

The wire rope was kindly furnished by the John A. Roebling's Son's Co., of Trenton, N. J., and by the Washburn and Moen Mfg. Co., of Worcester, Mass. Two kinds were used, both with hemp centers:

(a) The ordinary lay in which the center is composed of a strand of hemp or wire around which are twisted six strands, each strand consisting of seven wires.

(b) The lang lay in which, unlike the ordinary lay for hemp and wire, the wires forming the strands and the strands comprising the rope are all laid in the same direction. By this method a longer continuous surface of any one wire is exposed to wear, and the crowns of the strands are less pronounced, thus reducing the cutting tendency.

The impact machine was made from an ordinary wooden pile driver. The hammer was attached to the lower end of the wire rope and a heavy plate to the upper; the plate, including the rope and hammer, were raised with the hoisting rope as usual using a hand crab; the plate was tripped at the proper height allowing all to descend freely until the plate was caught by cross blocks and the hammer by the tension of the rope. These cross blocks were formed of two pieces of oak on each side of the upright guides of the driver, each 4″ x 14″, firmly bolted together; on top of these were two bars of steel, each 4″ x 6″ x 25″. The oak blocks were supported by four 6″ x 8″ x 16′ hemlock posts resting on the driver sills and bolted to the uprights. The wire rope at each end passed through a conical hole in a cast block 10″ x 12″ and 4″ thick, the larger diameter of the hole being next to the end of the rope. The lower block was attached to the hammer, while the hoisting rope was attached to the upper block by means of a tripping apparatus; two horizontal pieces extending between the guides of the driver and attached to the upper block kept its motion vertical and insured its striking the cross blocks centrally when tripped.

The wire rope was attached to the block by extending the end through about four inches, then unstranding and bending each wire over so that the ends projected back into the large end of the conical opening and filling the voids with melted solder or babbit metal. Benzine was used to clean the oil from the wires, and then soldering fluid to insure adhesion between the wire and metal. Experiments were made which showed that the wire was not heated sufficiently to affect the temper. It is suggested, however, that Wood's metal would be preferable to solder, as it would greatly facilitate the insertion and removal of the wire. The holes in the blocks were large enough for the largest sized rope, and conical brass bushings were used in connection with the smaller sizes. The tensile tests were made in a 400,000-lb. Richlé machine.

IMPACT TESTS.

No. of Test.	Rope.					Drop of Hammer.	Weight.	Blow in inch-lbs.
	Make.	Lay.	Diam.	Wt. per Foot.	Clear Length.			
1	A	Ord.	0.5″	0.37	10.3′	14″	1673	23,422
2	A	Ord.	.5		10.9	14		23,422
3	A	Ord.	.75	.85	11.1	36		60,228
4	A	Ord.	.75		11.0	36		60,228
5	B	Lang.	.5	.30		13		21,749
6	B	Lang.	.5		10.9	13		21,749
7	B	Ord.	.5		10.9	12.5		20,913
8	B	Ord.	.5	.34	10.9	13		21,749

The weight of the upper block	116	pounds.
The weight of the lower block	103	"
The weight of the hammer	1570	"
The weight giving blow on rope	1673	"

The friction on the guides was neglected.

NOTES.

No. 1. First tested with 10.5-inch drop; then with 12-inch, when rope pulled through upper block. Clear length 11.2 feet. Six wires broken. Rope shortened 10 inches; refastened to upper block; 12-inch drop left it in apparently good condition, and 14-inch broke 32 wires near the lower block and the other 10 pulled through.

No. 2. Drop 12 inches tried, then one of 13 inches; good condition. With 14 inches two strands (14 wires) broke near the lower block, and 7 wires near the upper block

No. 3. Drops of 24, 26, 28, 30, 32, 34 inches were tried. With 36 inches four strands broke near the lower block.

No. 4. Drops of 34 and 35 inches tried. With 36 inches two strands broke near the upper block.

No. 5. Drop of 12 inches tried. With 13 inches all the wires broke near the upper plate and a few near the lower.

No. 6. Drop of 12 inches tried. With 13 inches three strands broke in the upper block.

TENSION TESTS.

	A. Ordinary Lay.								B. 0.5″ diam.					
	0.5″ diam.		0.5″ diam.		0.75″ diam.		0.75″ diam.		Lang.		Ord.		Ord.	
Load in lbs.	Stretch per ton in 32″.	Total stretch in 32″.	Stretch per ton in 31″.9.	Total stretch in 31″.9.	Stretch per 2 tons in 38″.7,	Total stretch in 38″.7.	Stretch per 2 tons in 39″.4.	Total stretch in 39″.4.	Stretch per ton in 37″.	Total stretch in 37″	Stretch per ton in 37″.8,	Total stretch in 37″.8.	Stretch per ton in 38″,2.	Total stretch in 38″.2.
350			″.000	″.000					″.000	″.000	″.000	″.000	″.000	″.000
600					″.000	″.000	″.000	″.000						
1500	″.000	″.000										.096		.068
2100		.033		.060						.126	.090	.186	.067	.135
4100	.050	.083	.048	.108		.026		.051	.130	.256	.068	.254	.083	.218
6100	.053	.136	.039	.147					.090	.346	.094	.348	.092	.310
8100	.048	.184	.080	.227	.054	.080	.084	.135	.141	.487	.098	.446	.116	.426
10100	.057	.241	.047	.274					.279	.766			.173	.599
12100	.106	.347	.086	.360	.069	.149	.040	.175						
14100	.104	.451	.121	.481										
16100			.305	.786	.018	.167	.057	.232						
20100					.065	.232	.067	.299						
24100					.097	.329	.079	.378						
28100					.090	.419	.090	.468						
32100					.093	.512	.126	.594						
36100					.210	.722	.184	.778						
40100							.375	1.153						

No. 7. Drop of 12 inches tried. With 12.5 inches two strands broke near the upper clip.

No. 8. Drop of 12 inches tried and a snap was heard which led to the suspicion that several wires were broken in the block. With 13 inches 3 strands broke near the lower block, while the remaining ones were held in the solder.

The breaking loads were: No. 1, 15,050; No. 2, 16,830; No. 3, 39,750; No. 4, 40,870; No. 5, 12,460; No. 6, 11,970. No. 7, 12,860.

In using the elongation per unit load from the above table to compute the maximum tension or breaking stress under impact, allowance should be made for the elasticity of the supports, made up largely of timber, as already described. They would, however, compare favorably in rigidity with those met with in the use of ropes on derricks, hoisting engines, etc.

THE PRESENT AND THE FUTURE PROSPECTS OF GAS FOR LIGHTING, HEATING AND POWER PURPOSES.

BY HENRY R. LORDLY, C.E., '93.

When the electric light came into the field as a competitor of gas the gas companies were, generally speaking, in a prosperous condition but they were unprepared to meet the new competition, where improvement in lighting was demanded, and the fact that no new device was then in view to accomplish such improvement it seemed for a long time that gas as a illuminant must go behind. The stocks of many companies fell rapidly ; directors became doubtful and managers were helpless and all came to the one conclusion that the field for gas, outside of lighting, must be enlarged.

Affairs were worse in America than across the water. In Great Britain and the Continent the increased use of gas for heating purposes had brought about relatively greater consumption, and consequently cheaper rates, thus placing it in a better position to meet competition.

This no doubt accounts for the fact that the electric light has up to the present time only made slight progress in those countries compared with its success here.

A visit to many of the gas companies or works in those days is said to have illustrated quite clearly that those in charge were not always the best posted. Years of plenty had made them embryo princes of monopoly and the "good enough" system seemed to reign supreme. The managers of some works had certain rule of thumb methods of their own ; photomotors and other laboratory apparatus were not considered of much importance and while this may have done well enough under the old condition of affairs it was not assuring under a regime of stiff competition. The cost of gas must be lowered and the consumption kept up, or still more, must be increased, and in addition to this the residuals or bye products must be more carefully looked after.

For a time after the electric light made its appearance some progress was made in gas heating and motive appliances but little improvement was made in the lighting branch although many new devices had been patented. A few of these burners showed some merit, particularly those aiming at incandescent results by the aid of auxiliary burners or attachments, but only one reached any degree of perfection and that one may well be said to be the salvation of gas as an illuminant in the fight against electricity.

This burner is the one invented by Dr. Carl Auer von Welsbach of Vienna, Austria, now so well known as the Welsbach or Auer light. Dr. Auer found that by dipping a vegetable fibre in a solution of certain of the rarer earths, so called, and then burning this fibre, there remained an ash, conforming to the original form of the material dipped, which, upon coming into conjunction with the flame of a Bunsen burner, gave out a brilliant light. This is clearly explained in the specification of his first patent, granted by the German Patent Office. April 15th, 1887, which reads as follows :

"Oxide of Lanthanum, oxide of Yttrium and other rare earths combine in the most different proportions with magnesia or oxide of Zirconium when violently heated in a state of molecular mixture, forming peculiar substances, the properties of which cannot be derived from the properties of the ingredients. A chemical combination of the ingredients has been effected thereby.

These substances are characterized by an enormous capacity of emitting light, and by a high refractoriness when heated in a non-luminous flame."

He then gives a description of the compounds of zirconia with the rare earths ; also the compounds of the oxides of lanthanum, yttrium, zirconium and magnesium and then tabulates the following results.

" Among the magnesia compounds of the rare earths, the following composition has given about the best results :

60 per cent Magnesia (Mgo)
20 " " Lanthanum (La_2O_3)
20 " " Oxide of Yttrium (Y_2O_3)

Among the zirconia compounds the following :

60 per cent. Zirconia ($Zr O_2$)
30 " " Oxide of Lanthanum
10 " " " " Yttrium.

or

50 per cent. Zirconia
50 " " Oxide of Lanthanum.

When the amount of oxide of yttrium is increased, the light becomes more and more yellowish white, without losing its intensity."

After dwelling more particularly on these compounds of the rare earths the specification takes up the subject of the fabric or mantle as follows :—

"It is important, as well for the illuminating power of the incandescent burner and the utilization of the heat produced by the flame as for the stability of the incandescent network and the perfect steadiness of the light in a slight current of air to give to the "earthy tissue" a shape enveloping the flame and nearly conical. This end is attained, in accordance with the practical purposes, without special manual dexterity by the following process :"

"A fabric (preferably of vegetable fibre), the threads of which have about 0.2 mm. thickness, is previously cleansed with muriatic acid and well washed and then thoroughly saturated with a 30 per cent. aqueous solution well mixed by shaking—of the nitrates or acetates, finally well pressed and dried. The following dimensions are meant for a Bunsen Burner, having a mouth of 1 cm. diameter burning with a distinctly formed blue cone and consuming 70 liter gas per hour. (For larger burners, the dimensions are to be increased accordingly).

The impregnated tissue is cut in strips 10 cm. long and 10 cm. broad and plaited in little folds following the breadth, so that the length of the plaited tissue amounts to about 4 cm. Then a platinum wire about 0.2 mm. thick is drawn through the meshes of the upper border of the network thus formed, and bent to form a loop of 1 to 1 ½ cm. diameter : the ends are twisted together. The edges of the now tubular tissue which touch each other are sewn together with an impregnated cotton thread. Then the little platinum loop is fastened to a somewhat thicker platinum wire, a few centimeters long. After this preparation the mantle is kept ready for use. When producing on a large scale, it is better to use tissue of tubular shape woven by machinery."

He next describes the placing of the mantle over the burner and the incineration of the mantle in its first shape. After the burning off the earthy residue, so called, assumes the shape of the original web.

In a subsequent patent issued to Dr. Auer in Dec. 1887, he adds an additional rare earth or substance, viz : the Oxide of Thorium, and enumerates a new list of mixtures which give better results than those previously stated.

The new list is very interesting and will repay anyone for studying out carefully. It will be found in the Dec. 15th., '97, number of the *Progressive Age*.

The Auer or Welsbach light or burner as it is commercially known to-day is gotten up in various styles—that is the brass work and the shades used—most of which are so well known that it is unnecessary to mention them here. One shade will be treated on later.

As might well be imagined from reading the patent specification the light medium—the mantle—is of a frail construction and this weakness at first seemed likely to imperil the success of the new burner. In fact it was several years before such mechanical success was attained as to make it commercially useful. To-day the number of these lights in use must number several millions—three or four at least—and the increase is indeed a marvelous one.

The frailty of the mantle has been largely overcome by various mechanical contrivances although the mantle itself is much stronger than formerly and its strength increases with the length of time in use. Heat tempers and toughens it. In articles published now and then in the technical journals we read that so and so, has made certain tests of the Auer mantle and that he finds that it has a life of 600 hours, 800 hours, or 1,000 hours and so on as the case may be, but these tests, while quite interesting, are generally misleading and give no correct idea of the commercial value of the mantle. While there are cases where the mantle is subjected to great vibrations that, without specially constructed burners, it cannot stand long, in ordinary household and store use, if not injured by a body striking it, it should last easily a year out.

In Canada the Auer—Welsbach—light is managed by the companies there on a rental system which allows of a complete record being kept of each light in use and the writer has been in a position to obtain certain data as to the length of the life of mantles as will make certain laboratory tests, already mentioned, appear quite ridiculous.

He has been able to see records that give results of many thousand mantles—not merely a half dozen mantles—which have been used under all the circumstances imaginable. Below will be found a list taken from nearly one thousand lights in use ; this list not being a specially selected one you may judge from it what the Auer mantle can accomplish to-day when it is properly looked after.

The Canadian companies simply give the following instructions for obtaining the best results results from the burner, viz : "Let It Alone" and the record given shows what they accomplish by the assistance of careful and specially trained fitters and inspectors.

This list has not been published before and will no doubt surprise some who have mistaken ideas of the durability of the mantle.

Of course treated carelessly it is easily injured but when under such supervision as placed upon it by the Canadian companies it is not hard to see why the Auer light has been so successful there and how it has regained from the electric companies many consumers that at a former period belonged to the gas companies.

RECORD OF THE LIFE OF AUER MANTLES, AS TAKEN FROM THE FIRST 24 PAGES LEDGER, PLAN NO. 1. MARITIME AUER LIGHT CO. L'T'D. FEB. 1, 1897.

10	Mantles	lasted an	average of	24.3	months in	Residence,	and still	in use.
12	"	"	"	22.5	"	"	"	"
26	"	"	"	22.0	"	Church	"	"
6	"	"	"	21.6	"	Residence	"	"
27	"	"	"	20.2	"	Church	"	"
41	"	"	"	19.9	"	"	"	"
11	"	"	"	19.8	"	Office	"	"
12	"	"	"	19.5	"	Residence	"	"
33	"	"	"	19.3	"	"	"	"
7	"	"	"	18.5	"	Store	"	"
12	"	"	"	18.3	"	Residence	"	"
7	"	"	"	18.0	"	Store	"	"
6	"	"	"	17.3	"	"	"	"
11	"	"	"	16.5	"	"	"	"
19	"	"	"	16.4	"	"	"	"
8	"	"	"	16.2	"	Residence	"	"
6	"	"	"	16.0	"	"	"	"
21	"	"	"	15.0	"	Store	"	"
21	"	"	"	14.9	"	"	"	"
11	"	"	"	14.6	"	Residence	"	"
8	"	"	"	13.7	"	"	"	"
15	"	"	"	13.2	"	Store	"	"
90	"	"	"	13.0	"	"	"	"
9	"	"	"	12.7	"	Residence	"	"
30	"	"	"	12.0	"	"	"	"
9	"	"	"	11.6	"	"	"	"
34	"	"	"	11.3	"	Store	"	"
44	"	"	"	11.5	"	Residence	"	"
21	"	"	"	11.1	"	"	"	"
70	"	"	"	11.0	"	Store	"	"
21	"	"	"	10.9	"	Residence	"	"
24	"	"	"	10.7	"	Store	"	"
56	"	"	"	10.4	"	"	"	"
10	"	"	"	10.2	"	Residence	"	"
41	"	"	"	10.1	"	"	"	"
30	"	"	"	9.9	"	Store	"	"
21	"	"	"	9.8	"	"	"	"
37	"	"	"	9.7	"	"	"	"
14	"	"	"	9.4	"	"	"	"
9	"	"	"	9.2	"	"	"	"
12	"	"	"	9.1	"	"	"	"
24	"	"	"	9.0	"	"	"	"
27	"	"	"	8.6	"	"	"	"
9	"	"	"	8.0	"	"	"	"
16	"	"	"	7.5	"	"	"	"
988								

This shows an average life of 13 months and 9 days, and takes in all the various circumstances under which a light can be placed.

Leaving out the churches and allowing an average daily consumption of only three hours it will be seen that the average attained at date of record was over twelve hundred hours for say at least nine hundred lights. Some of those in the first hundred went as long as twenty-seven months and were used fully an average of three hours per day.

Using an anti-vibrating attachment some lights have gone for over a year in a cotton factory, while some have lasted equally as long in printing offices in rooms under the presses, and 96 lights in a woolen mill only required an average of one and a half mantles each to run them one year without any special fixtures.

The Auer light is more than an able competitor of the incandescent electric light where the conditions of economy and effective light are considered. This can be better shown by results than by statements, and the records of the Martime Auer Co. of Canada show that in St. John City at least 250 stores that formerly used the electric now use the Auer light. They also light twenty-four churches. In Fredericton, Moncton and Yarmouth, all small clties in Maritime Provinces the electric light has almost been entirely wiped out of stores and the arc light has suffered almost as badly as the incandescent. This, too, in places where a few years ago the gas companies were looked upon as deceased enterprises.

The writer will make no statement concerning other places except that he knows there are many other American towns that can show as good results, and will merely add that the prices of electric current in the above towns has been relatively low—50 to 60 cents per month for 16 c.p.—while gas runs from $1.75 to $2.50 per thousand cubic feet.

After having made clear its claim to a share of the indoor lighting the new gas burner has turned its attention to the outdoor field and under the supervision of a specially controlled company the Welsbach Street Lamp has been making a bid for outdoor illumination.

In Great Britain and Europe the use of these lights on the streets is quite extensive but they have not the electric arc to contend with so when in America it was proposed by some to employ the same means for street purposes, in competition with the arc light, it was not remarkable that many doubted the results.

Why ! said some, how can a sixty candle power light do as well as a two thousand one ? They forgot for the moment, or did not know, that there really is nothing, on our streets at least, that approaches two thousand candle power, and the writer ventures to say that there is not one light out of a hundred, in any city on this continent, that gives over fine hundred effective candle power and the majority of them do not give three hundred candle power. By effective candle power he means measured, the same as in ordinary photometry, in one direction only.

To prove this statement he points out the fact that four Welsbach lights properly placed on a street will seldom ever fail to give a more efficient and a more equally distributed light than one electric arc placed right in the center of the four. He has found in a great many cases in his experience that four Welsbach or Auer lights will light most any store better than an arc light and he has never had to use more than six.

Specially constructed lamps and burners are employed in street lighting and when erected on nicely painted lamp posts they give a much neater appearance than the cumbersome poles employed in the electric system.

Below is a cut of a lamp now being employed by the Welsbach Street Lighting Company of America. This lamp is now used in nearly fifty cities in the United States, in whole or part of their street system, and the following cities are noticed in the list :

Worcester, Cambridge, Brookline, Mass., Freehold, East Orange, N.J., New Haven, Conn., Baltimore, Md., Beaver, Pa., Williamstown, Mass., Prospect Park, Brooklyn, Western Boulevard, N. Y., Ithaca, N. Y., and tests are now being made in many other cities.

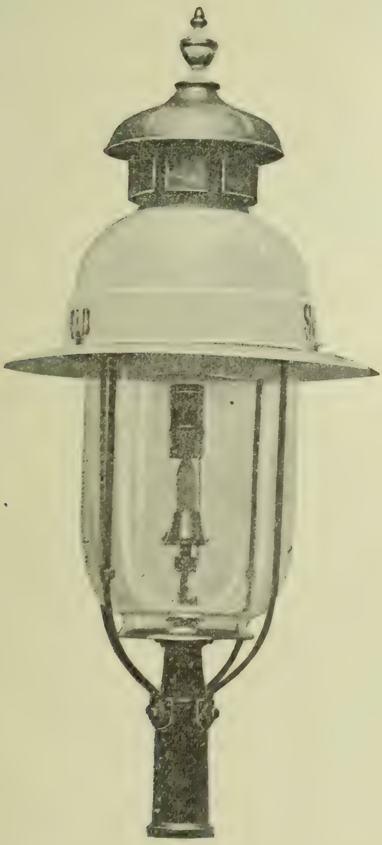
On streets having shade trees the incandescent gas burner has an additional advantage.

In the present age of scientific advancement it hardly seems safe to predict what the future is likely to bring forth but it is certainly safe to say that the outlook for gas for illumination purposes is brighter than what it was some few years ago. In fact an examination of the stock exchange lists of illuminating companies will show that gas companies as a rule stand higher than electric lighting companies.

There will no doubt be changes and improvements in lighting devices, both for gas and electricity, but it must be remembered that the improvements to be useful must bring higher efficiency, hence economy, in lighting. No light, however beautiful, will attain commercial success unless it is economic.

The one weak spot in the incandescent gas burner, the method of igniting, will soon be improved. Self lighting attachments that will operate on the turn of the hand, the push of a button, will before long doubtless be on the market at a figure that will allow of universal use and then will the gas companies be on equal footing with their electric competitors in possessing a light of equal handiness. They already possess a light which is easier to read by and less injurious to the eyesight than the electric has ever proven to be.

In concluding the lighting branch of this paper let us give a few moments to one of many globes that have been particularly designed to improve the effect of the incandescent gas light. This globe will prove doubly interesting because it is a purely scientific one and allows considerable scope for those who are well up in mathematics and physics.



This globe which bears its peculiar name because it is wholly luminous is made of thick glass and is cut in a remarkable way. On the outside are grooves, with an upper edge horizontal and the lower edge cut at an angle of about forty-five degrees. The grooves vary in depth from the top down being deeper at the top. On the inside there are vertical grooves evidently with a sharper angle and the whole construction gives a very beautiful effect when surrounding a light of high intensity.

It is of additional value when in use with the Welsbach light because it has the effect of throwing the rays downwards as the rays of this light, when naked, are largely horizontal. It has been tried with good results in Europe for street lighting and in this country has grown quite popular for various uses.

For a full description, showing curves of efficiency, see *Progressive Age* for March 15th.,'97.

Competition has been fruitful in causing more attention to be given to the production of special appliances for the comfort of the consumer and the assistance of the gas companies. Special lines of globes, burners, self lighting attachments, pressure gauges, burner testers, prepayment meters, complaint meters, universal pendants, etc., etc., have all been subjects for patents and some of these are interesting and valuable from many points of view. The Complaint Meter is worthy of attention and it will doubtless be found a valuable adjunct to gas companies.

Under the top of the meter is placed a revolving drum, same as in a chronograph machine, which drum contains a record bearing paper. The drum revolves by clockwork and runs for one week. When the meter is not being used, or when there is no leakage, the recording pen makes a straight line, but as soon as the gas is lighted the little pen makes tooth like marks, beginning from the hour that the gas is turned on or ignited and continues to indicate until the gas is shut off again.

With such an instrument as this a gas company can soon convince a fault finding consumer, who according to his own statement never uses the amount he is billed for, just what he really does do, and in this way may be able to remove a large amount of the habitual complaints that some customers make about their bills.

When a complaint is received, the company, unknown to the consumer, would simply place one of these meters in, and at the end of the week would be able to inform the consumer whether his or her idea as to the hours which light was used is correct or not. The meter also detects leaks and will be useful for that purpose too.

The meter described above is made by the Maryland Meter Co., Baltimore.

COOKING BY GAS.

The use of gas for cooking purposes has become so common now in Great Britain and the Continent, particularly in the larger cities, as to cause but little comment. It is almost the rule, not the exception as it is or was in many of our cities. However the past few years has brought about a radical change in its use for this purpose in America and the large output of gas ranges from American manufactories is ample proof of the progress that is being made here.

Economic gas ranges are now possible and thousands of them are in daily use, both winter and summer, and the number is daily increasing.

The pre-payment meter, (penny-in-a-slot) has aided the use of gas among the poorer classes in tenements in London and other large cities across the water and its adoption in this country will bring about the same result. By this meter the consumer can buy gas when and in what quantities he wants it, the gas companies are not put to the expense and trouble of booking small accounts, many of which they might lose and both sides are greatly benefitted by the system. In fact its use is not confined at the present to the poorer classes as there are many who prefer to pay as they go.

Cooking stoves, or ranges, put in free by gas companies and rented for a small fee is a method adopted by some companies to gain consumers and one company in which the writer is interested is now doing this and finding it a successful method.

Probably the branch in which gas engineers are most interested now is the use of gas

FOR HEATING PURPOSES

and the field here is one that will give ample scope for much study on their part.

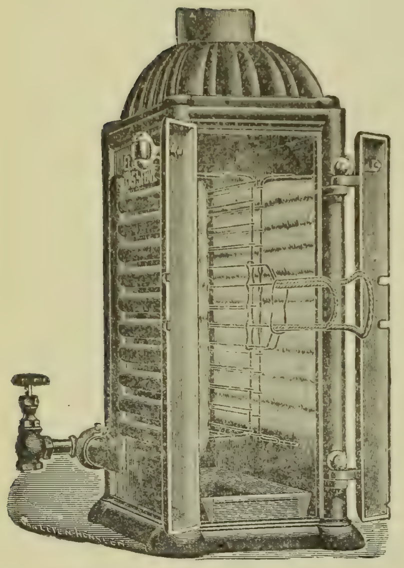
Much improvement has been made in appliances for this particular work, and some are fairly economic, but there is a demand for something that is as cheap as coal and when this is approached to a somewhat nearer degree than at present provided, it will greatly enlarge the gas sales. Special furnaces, both for hot air and hot water, are now constructed, but in the writer's opinion the hot water system will eventually turn out the most successful. Some English makers are turning out a hot water apparatus which is very efficient for heating conservatories and small apartments and no doubt before long the system will be sufficiently improved to allow its adoption for general purposes at no great increase over the cost of coal.

On another page will be found cuts of a gas log, gas radiator much used for auxiliary heating, and also a gas range and a steak broiler which are greatly used at the present time, besides which there are a multitude of other heating and cooking appliances.

A heater that is worthy of more than passing attention is manufactured in this country and is purely an American invention.

Gas Log—American.—
For placing in Grates.

Cooking on
An American Gas Range.
Consumes from 500 to 700 cu. ft.
per week for average family.



Steak or Meat Broiler.
An American Pattern.

American Gas Radiator.
Heated a room 12′ × 13′ × 8½′ from 40$^1/_0$° F. to 60° in 57 min. with 20 ft. of gas of 18 c.p. Ooutside temperature below zero. Test by H. R. I.

It is known as the Backus Patent Radiating Steam Mantel and Fireplace and for beauty, efficiency and economy, surpasses anything that is on the market at the present time. Perhaps, because it is more expensive to purchase than other heaters, it is not as well known or as much used as common heaters but it must only be a matter of time when it will be.

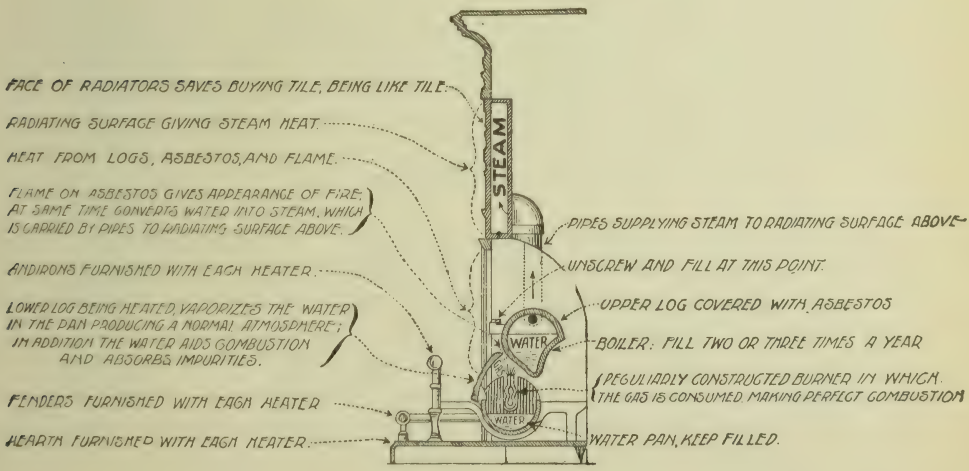
The mantel is constructed of cast metal and resembles the decorative wooden mantels as now generally used although probably somewhat deeper. The fireplace has imitation logs one of which contains water. From this log to the interior of the mantel—the latter being hollow—a pipe connection is made. When the gas is lighted the water is transformed into steam and passes all through the heart of the mantel which now acts the same as a steam radiator but has a very much larger radiating surface than most ordinary radiators.

Various sizes and styles are made and some of them will heat a room 20 x 20 x 12 ft., in zero weather, by the use of fifteen feet of gas per hour.

Great hygienic advantages are claimed for this heater which the writer cannot altogether verify at the time of writing but he believes that results equal to those claimed to have been obtained are perfectly feasible.

The cut below is of a section through the middle of the mantel and will explain very clearly the construction of the entire heater.

This heater is made by the Backus Co. of Williamsport, Pa.


FACE OF RADIATORS SAVES BUYING TILE, BEING LIKE TILE.
RADIATING SURFACE GIVING STEAM HEAT.
HEAT FROM LOGS, ASBESTOS, AND FLAME.
FLAME ON ASBESTOS GIVES APPEARANCE OF FIRE. AT SAME TIME CONVERTS WATER INTO STEAM, WHICH IS CARRIED BY PIPES TO RADIATING SURFACE ABOVE.
ANDIRONS FURNISHED WITH EACH HEATER.
LOWER LOG BEING HEATED, VAPORIZES THE WATER IN THE PAN PRODUCING A NORMAL ATMOSPHERE; IN ADDITION THE WATER AIDS COMBUSTION AND ABSORBS IMPURITIES.
FENDERS FURNISHED WITH EACH HEATER
HEARTH FURNISHED WITH EACH HEATER.
STEAM
PIPES SUPPLYING STEAM TO RADIATING SURFACE ABOVE
UNSCREW AND FILL AT THIS POINT.
UPPER LOG COVERED WITH ASBESTOS
WATER
BOILER: FILL TWO OR THREE TIMES A YEAR
GAS
PECULIARLY CONSTRUCTED BURNER IN WHICH THE GAS IS CONSUMED, MAKING PERFECT COMBUSTION
WATER
WATER PAN, KEEP FILLED.

Among the many contrivances for the use of gas in the household, outside of heating, besides those already mentioned, the instantaneous water heater is one that deserves consideration and in the absence of a cut the following description will assist in explaining one style :

Inside of a metal cylinder or upright vessel is placed a pipe coil having a number of small burners arranged to burn a blue flame. Above this coil, surrounded by the outer shell, is a truncated copper cone having a trough near the top at about right angles to its surface. Above this are two caps, or flatter cones, of copper which act as water sheds. Through these, and up through the center of a pipe of about three inches diameter, is carried the supply water which, on leaving the supply pipe, falls through a number of fan blades—like little wind mills—and is rapidly sprayed on to the caps or hoods.

When the gas is ignited the cone and the upper hoods or caps are heated, very quickly, to a high temperature. The water is turned on at the same time as the gas, and, falling upon the heated hoods, within a minute, a supply of hot water flows through the trough and out by way of the escape pipe.

This flow of water will continue as long as the gas remains lighted and when in operation, say in a bath room, the heater will supply sufficient hot water for a bath while one is getting ready for it. The water can be intensely heated by regulating the supply, and experiments show that the system is about the most economic and rapid method of heating water that exists.

With gas at about $1.25 per thousand cubic feet the average cost of a bath would be about one cent.

By the use of such appliances as these it is not hard to see that gas has a field, outside of illuminating, which will give gas companies a much better opportunity to increase their output than in the days when they depended on lighting alone.

GAS AS A MOTIVE POWER.

The use of gas as a motive power is one that probably interests most gas engineers to the greatest extent and this brings us to the consideration of the progress which has been made in this line since the invention of the early Otto make of gas engine.

The gas engine at one time, and not many years ago either, was looked upon as a motor good enough to run coffee mills and printing presses, but later developments have changed its field materially and now it is ambitious to compete with steam.

To-day gas engines are in use up to two hundred horse power and some much larger are under construction or in actual operation.

In view of the many articles on this subject of late, particularly in the *Engineering Magazine*, we will not dwell very long on it here except to say that, with the assistance of improved engines, gas companies hope still further to enlarge their field and indications point in a direction that makes it probable that their hopes will not be disappointed.

One of the new uses to which gas engines have been applied is the generation of electricity. Special gas engines are now made for running dynamos and it is a curious fact that some gas companies are now furnishing light in some cases by means of electricity generated by gas power. This is one way of meeting a competitor.

While the efficiency of the gas engine has been improved very much there have also been many mechanical improvements ; the engine is not so cumbersome as in its early days and American inventors have added one great improvement—the electric igniter.

The cut below is an American made Otto gas engine of 4½ h. p. with an electric spark igniter and is a most efficient machine.

The writer has experimented considerably with this engine and believes it can be run with a consumption of from 13 to 15 cubic feet of gas per horse power, per hour, and since it has been in operation for a while, it can be started with only a partial turn of the fly wheel.

This motor was adopted in place of a 3 h. p. electric motor and for the same class of work, the same number of hours, it was found to be only one-third as expensive as the former.

In Part I of Vol. XI, Canadian Society Transactions—Civil Engineers—will be found a series of lectures by Prof. Nicholson of McGill College, on "Transmission and Distribution of Power by Compressed Air, Fuel Gas etc.," and Professor Nicholson shows that "compressed air, and fuel gas, more or less compressed, will in the future become the most formidable competitor of electricity as the media to be employed by engineers whenever power has to be obtained from a natural source, transformed and transmitted for utilization to considerable distances from that source."

The newest use to which the gas motor has been applied is that of propelling street cars and in this connection it has proven much more satisfactory than the fondest hopes of the gas engineer looked for.[1]

The system was adopted at Dessau, Germany, two or three years ago and the work accomplished by it is indeed remarkable.

The motor is constructed on the Otto principle and is placed underneath the car, the gas being stored in cylinders the same as in the

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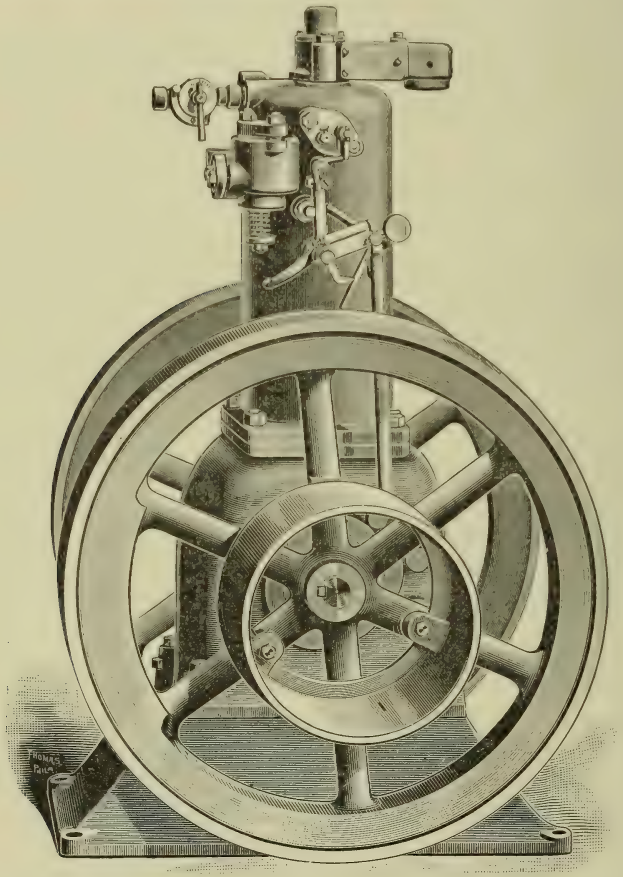
[1] See also Progressive Age, Sept. 16, '95, and Feb. 15, '96. The Engineer, July 17, '96.

Pintsch system of car lighting; you no doubt know that upwards of seventy thousand cars in Europe are lighted by gas under the Pintsch patents.

The gas is stored under pressure of about ten atmospheres and the cylinders are sufficiently large to hold enough for a run of seven miles and upwards. The system is really a development and improvement on the patents of Herr Luhrig, of Dresden, where a line is in operation, also at Berlin, and, since its adoption in Dessau, several other lines, in different towns have been constructed.


THOMAS PHILA.

American made Gas Engine, 4½ h. p. Spark Igniter.

The British Gas Traction Co., opened a road at Blackpool, July, '96, from Blackpool to St. Annes to Lytham, and so far it has been successful and has been extended considerably from what it was at the start.

The first cars weighed about seven and a half tons and were regulated to a maximum speed of 8½ miles with 14 h. p. A larger motor has a speed of 15 miles per hour.

One of the great advantages of the system is that each car is independent in itself. There is no central station to depend on and no laying up of the whole road in case of an accident to the power, which is not unfreqent with electric traction.

The following data, which the writer has been able to obtain, is taken from records made on the Blackpool Line and will give a good idea as to what can be done from an economic view.

Cost per Mile of Running Gas Tramcar :

Cost of Gas	1.05	pence
Driver	1.15	"
Depreciation, Repairs, &c	.80	"
Labor for above	.45	"
Cleaners	.10	"
Compressing gas	.15	"
Total	3.70	pence.

Or say seven and one-half cents which will be found, on comparison with other systems, remarkably cheap.

Gas tramcars are now being tested at both Paris and Amsterdam and are proving perfectly successful.

The British company recently opened a line at Trafford Park, Manchester, which runs to the Ship Canal and will be extended further. They are also carring on the line from Lythan to Fleetwood, a distance of 20 miles.

Nothing of importance has been done on this side of the water with gas traction outside of some experimenting.

The writer has been engaged in studying it for some two years and a company, of which he is engineer, has recently obtained a charter, through Act of Legislature, to build a gas road at Charlottetown, P. E. I., Canada, and four miles will soon be under construction.

The system is claimed to be unsurpassed in economy by any other motive power, particularly in towns where the traffic is not extremely heavy or where cars are not required to be in motion constantly. Besides the initial cost of plant is by far the cheapest.

Experience proves that street car companies often fail in small towns with the electric system as the expense of operating is practically the same whether there are many or few passengers to carry. In the case of the gas cars, when the car is idle, the only expense for power is the slight amount of gas required to keep the fly wheel moving and where a car is stopped for say a half hour or more the machinery may be also stopped.

The cars which will be used at Charlottetown can mount a gradient of one in twenty and will be able to pass over a curve of less than forty feet radius. When running at eight miles an hour the driver can stop a car within its own length.

Later on a special paper on this subject will be provided for your Transactions as the writer believes that the honor of introducing the system into this country, in a really commercial shape by the means of a regularly incorporated company, will belong to a Cornell graduate.

This paper started out to give an idea as to the " Present and Future of Gas," and before bringing it to a close it would be well to look into the relation of the civil engineer, or engineering graduates, with the gas industry, so called.

A recent English journal stated that the day for the old time gas engineer was over and the future, as well as the present, demands that only men of sufficient technical education be placed at the head of gas corporations. Only within a few months there were a number of positions as gas managers vacant in Great Britain simply because the proper men were not available and the writer believes that there are many companies in this country that could make a change in the right direction by putting men at the head of their business who were properly qualified engineers, or civil engineers.

The latter, he thinks, has a great advantage. An advantage in being able to make proper street plans ; in supervising the accurate laying of pipe ; in being able to design and construct truss work for retort houses and holders and, if the company is also interested in street railway operating, he can also act as engineer to it.

In addition to the above he should be able to test coal and analyze gases, as there is more of this work being required every year and to this he can give considerable time during the winter months. So you will see that the civil engineer is very closely allied to this business and any young engineer who has a liking to the lines enumerated cannot do better, in the writer's opinion, than get emloyment with a gas company and work up to the management of the same.

Many graduates will find, like those of us in the past found, that positions are not waiting for them all over the country, but that they must in a measure assist themselves in getting work, and the writer believes that gas undertakings will in the future furnish employment for many able graduates.

And now a word to the Engineering Faculty, to those who have so ably guided us through our four years of hard work.

Do you not think that the present status of the gas business requires more attention than was given to it in the days when the writer was an undergraduate? Do you not agree with the writer that there is a field for many of your graduates? and that there is in it as brilliant a future for someone as in bridge engineering or some kindred work?

Within a few months a former graduate wrote to the writer inquiring about work, not having met with very great success in the town where he was situated, and was totally unaware that at his very feet lay a position as manager of a gas company that he could have obtained by the assistance of a few friends. But, he claimed that he knew nothing of the business, and therefore lost a golden opportunity as his training and energy, compared with the former managers, would have been able to put the company right on its feet and given him a good salary too.

So the writer believes that a complete course of gas engineering should be provided for any students who desire it and that in providing it the College of Civil Engineering of Cornell University would be only adding to its strength.

Gas analyses, a more complete knowledge of chemistry, the theory and design of retort benches, regenerative and recuperative benches or furnaces, practical work in photometry, etc., would be the only extra studies required and a thesis on some important topic concerning gas would finish a very valuable course.

The writer trusts that this will soon be provided and only regrets that he is unable to do more toward it than to contribute a rather hastily prepared paper which he feels does but poor justice to such an extensive subject.

In conclusion he gives below a list of the leading gas journals and some of the books most used by gas engineers and wishes success to those whose names are about to be placed on the list of graduates and hopes that they will be true to themselves and to their Alma Mater.

JOURNALS DEVOTED TO ILLUMINATING GAS, ETC.

Progressive Age, 280 Broadway, N. Y. City; American Gas Light Journal, 32 Pine St., N. Y. City; Journal of Gas Lighting, 11 Bolt Court, Fleet St., London, England; The Gas World, 3 Ludgate Circus Bldgs., London, E. C., England; Journal de L'Eclairage au Gaz, 4 Rue de la Bourse, Paris, France; La Gaz, 12 Rue Fontaine, Paris, France; Journal du Gaz et de L'Électricité, 23 Rue de Navarin, Paris, France; Le Moniteur de L'Industrie du Gaz et de L'Électricité, 24 Rue Condorcet, Paris, France; Journal des Usines à Gaz, 65 Rue de Provence, Paris, France ; L'Industrie Électrique, 9 Rue de Fleurus, Paris, France ; Journal fur Gas Beleuchtung und Wasserversorgung, Glückstrasse 11, Munich, Germany ; Der Gastecckniker, Taubstummengasse No. 6, Vienna, Austria ; Zeitschrift für Beleuchtungswesen, Heiz und Lüftungs Technik, Bulostrasse 91, Berlin, Germany ; Zeitschrift fur das Gas und Wasserfach, Friedensgasse 4, Vienna, Austria ; Kraft & Licht, Düsseldorf, Germany.

Beside the above there are several journals, of recent birth, which are devoted to acetylene gas, which has not been touched upon in this article. Also a number of the leading technical journals devote considerable space to gas topics but generally reprint from the recognized gas publications.

The books that will be fonnd of especial value to the gas engineering student are :—

Chemistry of Illuminating Gas, by Norton H. Humphreys ; Gas Engineers Laboratory Handbook, by John Hornby, F. I. C.; Chemistry of Gas Manufacture, by W. J. A. Butterfield ; Inorganic Chemistry, by Prof. Victor van Richter ; Coal, Its History and Use, by Prof. Thorpe ; Construction of Gas Works, by Walter Ralph Herring ; Distillation of Coal Tar and Ammoniacal Liquors, by George Lunge ; Gas Works, Their Arrangement and Construction ; Practical Photometry, by W. J. Dibdin ; The Management of Small Gas Works, by C. J. R. Humphreys ; Manual for Gas Engineering Students, by D. Lee ; Ammonia and Ammonium Compounds, by Dr. R. Arnold ; Practical Hints on Regenerator Furnaces, by M. Graham ; Practical Handbook of Gas Engines, by G. Lieckfield : Practical Treatise on Heat, by Thomas Box ; Technical Gas Analysis ; Gas and Gas Works, by Hughes and Richards ; Gas Coals and Cannels, by David A. Graham ; The Incandescent Gas Light, by Wilhelm Gentsch, Progressive Age Pub. Co.

Any of these books may be obtained through the Progressive Age Co. or the American Gas Light Journal, New York.

SUBSTRUCTURES OF TALL BUILDINGS.—A REVIEW OF RECENT DEVELOPMENTS AND TYPICAL FEATURES.

BY FRANK W. SKINNER, '79.

The first preliminary of actual design and construction of the foundation of a tall building should be a most thorough and scrupulous exploration of the soil below its site and this can be effected with more or less accuracy in various ways. In soft soils soundings can be made by driving a steel rod to a considerable depth. In clay and other earths a well auger can be bored down sometimes a hundred feet or more and bring up excellent indications of the different strata encountered if rocks and stones are not met. Absolute data can only be secured by means of sinking trial pits or by making diamond drill borings and recovering the core, but as both of these methods are comparatively slow and expensive and cannot often be used, others are generally adopted, although the indications they give must be mainly considered as negative ones. The most common method is by means of sinking test holes by the jet system. In this process sections of pipe about two inches in diameter are driven down vertically by the combined action of a hammer and of an interior hydraulic jet at high pressure delivered through a three-quarter-inch inside pipe. These soundings can be made rapidly and to a great depth and if conducted with care and the effluent water and sediment preserved, valuable indications may be had of the character of the soil and the probable distance to hard strata. Single borings are, however, of very little use and likely to be misleading, but a considerable number of them can be compared, and especially if tested by one or two diamond drill borings, may be found adequate. Sometimes when the borings are conducted in the open air, a light pile driver frame, operated by hand or by a small engine, is provided for driving the pipes, but the borings are very often taken in the cellar of the old building, before it is demolished to make room for the new one, and here the head room is so small that different methods are required. Many borings of this kind have been made where the pipes, in sections from three to ten feet long, were mauled down by hand by two men, while a third man, with a hand pump, supplied the pressure for the hydraulic jet. In 1890 about two hundred testholes, with an average depth of 28 feet, costing $4.31 each, were sunk by the jet process by a gang of four men, using a hand force pump and pulling the pipes up with a lever and chain grip.

The bearing power of soils varies from swamps and quicksands that will not support a man's weight, to hard clay and fine sand, that will safely sustain several tons per square foot. An ordinary earth may be expected to carry from one to three thousand pounds per square foot, which suffices for buildings of moderate weight. But for very tall or heavy buildings, the pressures may exceed twenty thousand pounds per square foot or eight and one-half million pounds for one pier, as in the Manhattan Building, where a load of 8,553,000 pounds was carried on a pier 11 x 36 feet and the total weight of the building imposed a load of over 7,000 pounds per square foot upon the entire area included between its outside walls. Among the experiments made upon the compression and settlement of soils was that of clayey soil at the Albany Capitol which did not yield under a weight of 4,000 pounds per square foot, but showed adjacent upheaval when the load was increased to 10,000 pounds. At the Library of Congress in Washington the soil developed a resistance of about 20,000 pounds per square foot. In Paris, France, cast iron blocks, 20 inches square, set on the surface of the earth, sustained a load of 10,800 pounds per square foot; with 14,600 pounds they settled 11 inches in a few hours. Other tests made with masonry piers 6½ feet square settled much less than did a large embankment on the same soil sustaining one-half as great a load per square foot. The New York foundations of the Brooklyn Bridge piers transmit without perceptible settlement a pressure of nearly 14,000 pounds per square foot to the fine yellow sand that sustained a 12 x 12 inch timber loaded with 20,000 pounds per square foot without causing any penetration. In the business part of Chicago the surface of the ground is about ten feet above lake level and about five feet above the original surface and the upper stratum of loam, only ten or fifteen feet thick, overlies a hard, dry crust of clay about ten feet thick which is underlaid by from sixty to one hundred feet of soft, wet, unstable blue drift clay, which is supported on rock or hardpan. This clay is in a semi-plastic condition and has a great avidity for water; it contains beds of quicksand and is unable to sustain any considerable loads below the surface of the upper crust. If the clay stratum is loaded with about three thousand pounds per square foot and allowed to remain undisturbed for several months, the effect is generally to produce a uniform small settlement, which increases sometimes irregularly for a long time until it attains a total amount of perhaps two inches and ceases, after which considerable additional loading does not usually produce appreciable settlement or compression. The clay crust is considered capable of carrying safely a maximum load of about 4,000 pounds per square foot. The Chicago building laws permit it to be loaded to

3,000 pounds per square foot, and 4,000 pounds per square foot to be imposed on pure dry sand 15 feet thick. In New York, below the thin upper layer of made land and alluvium, the soil in the principal part of the business district generally consists of fine wet sand from 40 to 100 feet deep, underlaid by rock or very refractory hard pan and containing gravel, boulders and occasional pockets of quicksand. Eminent authorities consider the dense undisturbed sand, when it is free from quicksand, able to carry a maximum load of 8,000 pounds per square foot, which is permitted by the building laws, but 6,000 pounds is rarely exceeded and often causes perceptible settlement. Piles 14 inches in diameter well driven through this sand and spaced not more than 30 inches apart, are permitted by law to be loaded up to 40,000 pounds each. In Buffalo the building laws allow foundation loads of 7,000 pounds to the square foot on dry clay. In different cities the legal pressures permitted on different foundation materials are as follows : on concrete in Buffalo per square foot, 8,000 pounds ; in Chicago, 8,000 ; on cut stone in Buffalo, 14,000 ; in Chicago, 10,000 ; on brick in New York, 30,000 ; in Chicago, 25,000 ; in Boston, 30,000. Foundation piles are generally driven either to the rock, or to hardpan, or until the skin friction becomes so great that the penetration is not more than an inch or two for a blow of a one or two thousand pound hammer falling fifteen or twenty feet, and a pile which while being driven will be moved perceptibly by such a blow, will require a much heavier one to start it again after a short interval. This is illustrated by a record of some piles that, after being driven 35 feet through liquid mud and left undisturbed for ten hours, sustained a load of 13,000 pounds each before commencing to settle again.

It is customary, especially in Chicago, to make careful preliminary calculations for the probable amount of settlement for a building and to endeavor to secure uniform unit pressures on the foundations, so as to insure regularity in the settlement of the building as a whole. The Monadnock Building, 200 feet high, with a foundation pressure of 3,750 pounds per square foot, settled quite uniformly about five inches. The Old Colony Building, 215 feet high, with a foundation pressure of 3,220 pounds per square foot, settled from $4\frac{3}{16}$ to $5\frac{1}{8}$ inches in nine months. The Home Insurance Building, 159 feet high, is founded on a very thin crust of dry clay and, being started $2\frac{1}{2}$ inches above the required grade, eventually settled $2\frac{1}{4}$ inches, with only a variation of $\frac{11}{16}$ inch between any two points. A government building in Chicago settled from 6 inches to 18 inches during the first fourteen years after it was built, and the City Hall, in the same city, which was built on a monolithic concrete bed three feet thick, settled unevenly to a maximum of 14 inches. The St. Paul Building, New York, is 315 feet high and exerts a foundation pressure of 6,000 pounds per square foot on the surface of the fine dense sand upon which it rests. In preliminary tests to determine the bearing capacity of the soil, the surface was protected by a continuous thick bed of concrete through which a hole was cut just large enough to admit the end of a timber twelve inches square. The upper end of this timber was then loaded with a weight, gradually increased to 13,000 pounds, which only produced a settlement of $\frac{19}{32}$ of an inch after several weeks, during which the soil had been plentifully saturated with water. The actual foundation pressure, however, of about 6,200 pounds per square foot, only produced a settlement of about $\frac{3}{8}$ inch in a year. Notwithstanding these results, the owners of the adjacent property caused tests to be made of the bearing capacity of the soil underneath their building and, a larger area of the surface having been loaded, the settlement (the exact amount of which was not published) was such that they decided to excavate their cellar bottom simultaneously with that of the new building and to the same level. This was done with the view of reassuring them and preventing any unnecessary disturbance of the soil in case they should hereafter wish to rebuild on that site to a great height.

In the Marquette Building, Chicago, special provision was made to correct possible unequal settlement by providing an opportunity to insert hydraulic jacks under the bottoms of the columns at any time and lift them separately so as to adjust them by fillers of the required thicknesses. A similar but much more elaborate arrangement was constructed of beams and girders to support the columns of the St. Paul Building.

According to the character of the soil and the estimated loads to be imposed the foundations are of the surface, pit, caisson or pile types. As the loads are always large and movements of the structure are dangerous the foundations should be designed when possible so as to have no settlement or distortion whatever. Perfect stability can be assured by going to solid rock or hard pan or by preventing any disturbance of loose soils and restricting the pressures upon them to proper fractions of their bearing capacities. If neither of these methods is feasible the unit strains throughout the bearing surfaces should be made uniform and the foundations proportioned so that settlement should be regular and equal throughout.

Whatever kind of foundation is adopted the area of its base is many times greater than that of the column it supports. The offset masonry piers that formerly extended the bases of the columns and distributed their loads over the footings are almost universally superseded now by systems of steel beams and girders that are proportioned for the bending strains produced by the column loads. The heights of the piers are thus diminished and a large amount of space saved in the lower part of the building.

In the earlier examples of steel grillage foundations, the column load was often distributed horizontally by crossed layers of comparatively shallow beams, but the present practice is to make the grillage beams deep enough so that each can sustain its load without undue deflection when it is concentrated at the center and then to load these beams by a distributing girder proportioned to be stiff enough to receive the column load in the center and transmit nearly equal portions of it to each of the parallel grillage beams in the set. The simplest form is one column set in the center of a distributing girder which crosses the centers of several parallel grillage beams. Sometimes there are however two or more columns set on one distributing girder and sometimes two or more distributing girders are placed on the same set of grillage beams or one distributing girder may be supported by two sets of grillages. The grillages may rest on the earth or on tops of piers, which may themselves rest on the earth on piles, on grillages or on caissons.

One of the largest and heaviest distributing girders ever used in a building foundation is that in the Queens Building, * N. Y., which weighs about 74,000 pounds, and was shipped complete from the shops where it was built.

In the 12 story Franklin Building, New York, a maximum estimated load of 400 pounds per square foot of floor surface is carried on groups of 24 inch grillage beams that cover about 80 per cent of the area of the site, and they are proportioned for independent transmission of their respective column loads, and to transmit a pressure of 8,000 pounds per square foot to the surface of the fine gravel on which they rest, the concrete floor in which they are bedded is made continuous. The grillage beams are loaded by solid embedded distributing girders carrying one or two columns each. In cases where the dimensions of the distributing girders were considered inadmissible for single members pin connected trusses were substituted for them and their members reduced to convenient size and weight for transportation, were assembled in situ. Some economy of weight was also effected as the trusses weighed only about 60,000 pounds as against 80,000 pounds for a corresponding box girder.

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*This engraving and that of the pneumatic Caisson in the Standard Building and the Queens Building Floor are reprinted by permission of *The Engineering Record.*

Surface foundations have their footings at about the level of the cellar floor and generally consist of grillages of steel beams bolted together and embedded in concrete from 1 to 3 feet thick. Usually the sets of beams are arranged to support single columns or small groups of columns and are separated so as to act independently under their respective loadings.

Sometimes, however, the beams are bedded in a mass of concrete that is uniform over the whole area of the building, as is the case with the St. Paul and the Wilkes Buildings, N. Y., and the Spreckels Building in San Francisco.

The Spreckels Building is 19 stories or 300 feet high above the sidewalk and is 75 feet square. A pit 25 feet deep and about 100 feet square was dug for its foundations and the wet sandy bottom covered with a monolithic bed of concrete 2 feet thick. Upon this bed a layer of 58, 15-inch beams 96 feet long was set, and on top of them a cross layer of 63, 15-inch beams 91.5 feet long was laid, and concrete filled around them to their upper flanges. A solid base 4½ feet thick and 70% larger than the building was thus made and upon it were set the main columns that loaded it up to about 4,500 lbs per square foot. The 20 outside columns are anchored down to the lower tier of beams by pairs of steel bars.

The St. Paul Building is 26 stories or 315 feet high above the pavement and its foundations rest on fine wet sand just below the cellar floor 32.5 feet below the street and about 80 feet above bed rock. The bottom of the pit is covered with 1 foot of concrete upon which are set separate longitudinal rows of transverse 15-inch I beams with riveted and rolled longitudinal girders on top. Several columns are supported on each set of longitudinals by means of cross beams between their bases and the top flanges of the girders.

In Chicago most of the tall buildings have surface foundations of steel and concrete grillages, and as they are built over a very wet, soft, plastic clay that when exposed can sustain only small loads they are aptly called Raft foundations. Allowance is generally made for their settling 2 inches or more, and in the earlier buildings the grillages were made of crossed layers of rails. The Rookery, the Rand & McNally, the Masonic Temple and the Old Colony Buildings in Chicago have Raft foundations.

The World Building, N. Y., is 275 feet high and the loads from its interior columns are distributed over extended zones of the surface of the dense wet sand by means of inverted arches with concrete footings.

In the Wilkes Building, New York, the foundation consists of a monolithic concrete floor 27 inches thick, that is laid on the hard sand

and in which are embedded detached groups of 15-inch steel grillage beams that occupy more than half the entire area. These grillages are from 6 to 14 feet square. The smaller ones each support single columns whose fan shaped expanded bases serve as cross girders to transmit an equal part of the entire load to the center of each beam. The wall columns take bearing across the extreme ends of these grillages, which are symmetrically loaded by an interior column at or near the other end so as to produce a resultant through the geometrical center of the platform. At one corner the three wall columns are combined with one interior column and arranged with a distributing girder and a set of reaction beams so as to balance the loads on the grillage without obstructing the cellar space materially.

When conditions require it, pits may be dug with sheet piling or coffer dams, and the excavation carried down to a stratum sufficiently solid to sustain the footing from which masonry piers can be built to the cellar floor and there receive the column bases on beam platforms that distribute their loads over the top surfaces.

Where the bearing power of the soil is insufficient, piles may often be driven in it and, penetrating to hard strata below, transmit to it the foundation loads, thus acting directly as columns. Or, if the rock is too deep, the piles may, without touching hard bottom, develop sufficient skin friction to carry the load safely. Foundation piles are generally driven in separate groups with concrete two or three feet deep filled in the interstices between their heads. They are cut off below ground water level and capped with grillages of steel beams also bedded in concrete.

When the center of a column falls near or beyoud the outside of a pier, a girder may be supported on the center of a pier and overhang both sides of its bearing so as to carry the eccentric column at one end and have the reaction at the other end taken up by another column or columns. Such girders are called cantilevers and usually have both ends supported so as to be in equilibrium no matter what the moments of the column loads are. They are essentially cantilevers even when the bottom supports are extended so much as to make the reactions virtually distributed loads, so long as the pier is not continuous for the whole length of the girder.

The wall columns of the Exchange Court Building, New York, are carried on centilever lattice girders with reinforced end webs that have pin connections to heavy pedestals seated on the piers. Under each pier are from 16 to 40 12-inch piles that are loaded with about 17 tons each, and were driven about 20 feet through sand to hard pan, and to a refusal of ½ inch under a 2,000-pound hammer falling 25 feet. Some of the piers were built in pits sunk with sheet pile protection to a depth of about 40 feet below grade, or 10 feet below ground water level.

In the American Surety Building, New York, some of the pneumatic caissons are surmounted by two separate brick piers with a cantilever that has its main bearing on one of them and a second bearing on the other, but that overhangs so far that it was decided to give it additional security by anchoring it through the second bearing to a system of steel beams built into the bottoms of the pier, on the deck of the caisson.

A remarkable system of compound cantilever girders was adopted for the Gillender Building, New York, which is notable for its unusual proportions, or disproportions, of about 22 feet breadth by 74 feet length by an extreme height of 310 feet. The entire superstructure is supported on three wooden pneumatic caissons, each about 12 x 24 feet and 26 feet high, inclusive of the coffer dam. The caissons were sunk over 42 feet to penetrate a hard pan stratum, upon which they impose an estimated maximum load of about 9,000 tons, distributed over about three-fifths of the total area of the site.

The masonry piers are levelled off with concrete and on each one are set two grillage platforms. Across the ends of the grillage beams are set heavy cantilever girders, each of which overhangs the center of its support 5 feet at each end. Across each end of this first and lowest pier of cantilevers is another cantilever that overhangs the center of its support about 3 feet at each end. On these upper cantilevers are set one of the three groups of four columns, at the corners of a rectangle about 14 x 26 feet.

The base, measured between centers of grillage bearings, is about 10 x 15 feet. After the lower sections of the columns were set their bases and all the metal work below were bedded in a solid mass of concrete.

In the American Tract Society Building, New York, wall columns are carried excentrically from their piers, and the latter are symmetrically loaded by means of cantilever trusses arranged in pairs, so that while the wall column is seated on top of them one or more interior columns pass between them to separate piers of their own, which sustain any excess of weight not needed to counterbalance the wall column. The interior columns have reaction bearings riveted to their sides crossing the tops of the cantilevers, where they take bearing for any upward thrust.

The Park Row Building, N. Y., is the tallest in the world, being 390.75 feet high above the pavement or 424.5 feet from bottom of piers to top of dome. Some of its columns are loaded to a maximum of 3,000,000 pounds each. The most of the interior columns have separate piers, and most of the side wall columns are each balanced by a single interior column at the opposite end of a multiple webbed plate girder that distributes the total load upon platforms of beams on top of the masonry piers. All the piers have 12 inch concrete footings, that cap the top of short piles driven in the deep dense fine sand, and calculated to receive a load of about 32,000 pounds each. The outlines of the building are very irregular and in two places a pair of wall columns were made to balance each other at the ends of a double cantilever girder which was in turn supported upon a distributing girder that it overhung on both sides.

Where surface or pile foundations are inadequate or unsatisfactory, very heavy loads may be carried to rock or hardpan by the caisson process when the soil is so wet or treacherous that open excavation is impossible.

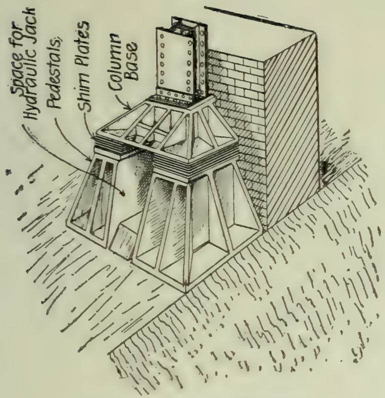
Pneumatic Caissons can always be used up to a depth of 100 feet but they are comparatively slow and costly. Hydraulic Caissons can only be used to advantage in fine loose soil free from boulders and obstructions and may sometimes endanger adjacent buildings by causing the quicksand to flow out from under their foundations and undermine them. The principle examples of Pneumatic Caisson foundations are the Manhattan, the Surety, the Commercial Cable, the Empire, the Washington Standard and the Gillender Building in New York. Wooden caissons were used for the latter, steel ones for the others.

The Johnson and the Meyer-Jonassen Buildings in New York are built with foundations carried down 40 or 50 feet through quicksand by Hydraulic Caissons.

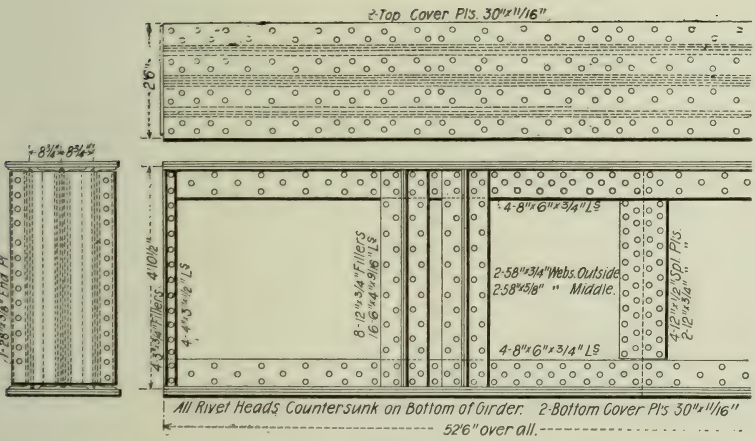
The pneumatic caissons themselves though primarily designed for constructional purposes, eventually become a part of the permanent substructure. They simply consist of a thin plate shell stiffened with interior braces and very heavy deck beams They have a reinforced cutting edge and usually have a working chamber about 7 feet high. Excepting the shape which is rectangular, circular or rounded in plan, and some minor differences in details those above mentioned were all of similar construction.

In New York the tall buildings have two or three stories below the level of the street and the excavations are sometimes 40 feet deep and 12 or 15 feet below permanent ground water level in a wet sandy soil. Special provision is made to resist exterior hydrostatic pressure and to prevent leakage. In the Commercial Cable building the concrete walls 51 inches thick were reinforced by steel riveted trusses in horizontal planes embedded in the solid masonry and reaching from end of side wall to end of side wall. The concrete floor, 44 feet below street level and 4 feet thick, was similarily reinforced against possible upward pressure by horizontal steel girders embedded in it and secured to the foundation piers. The inside of the wall was lined with ¼ inch steel plates and a multiple layer of felt laid in hot tar was brought down between the concrete wall and an outer brick facing and carried continuously across the bottom in the center of the concrete.

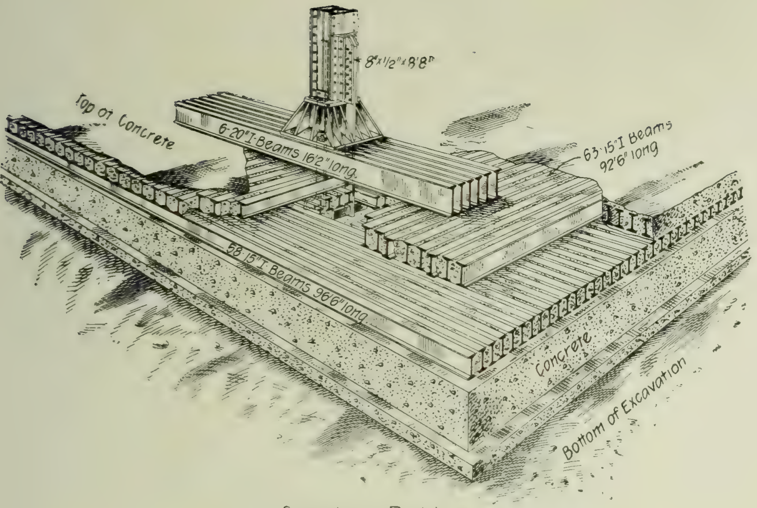
In the Queen's building, New York, the deep cellar has a floor consisting of a monolithic mass of concrete 56 inches thick enclosing a thick sheet of asphalt water proofing. The lower section of the concrete surrounds the heads of the groups of foundation piles, and in the upper section are bedded the sets of grillage beams that receive their column loads through enormous distributing girders that are partly bedded in the concrete.



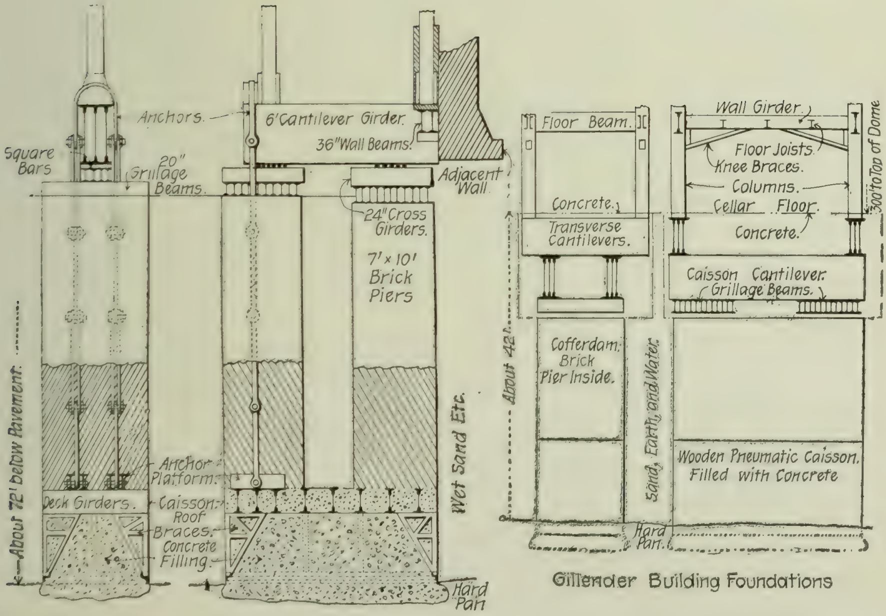
Column Adjustment
Marquette Building



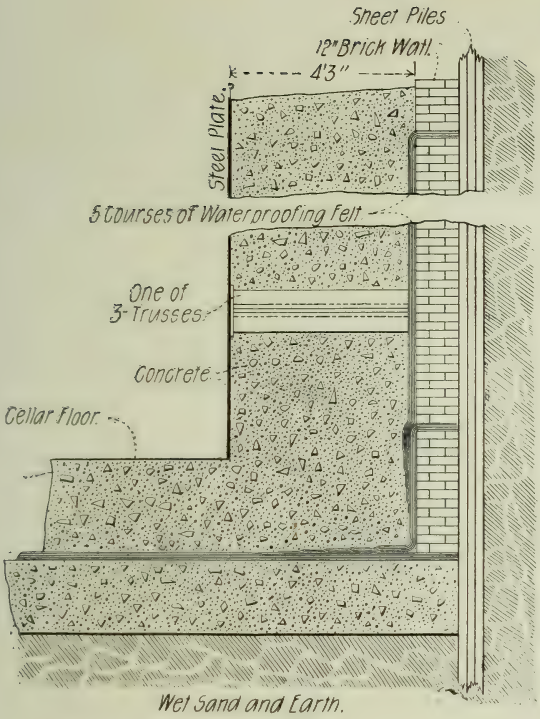
Queens Building


8"x1/2"x8'8"
Top of Concrete
6-20" I-Beams 16'2" long.
63-15" I Beams 92'6" long
58 15" I Beams 96'6" long
Concrete
Bottom of Excavation

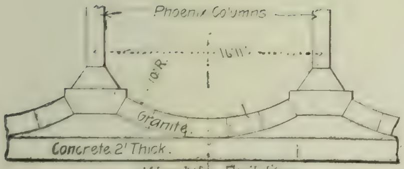
Spreckels Building



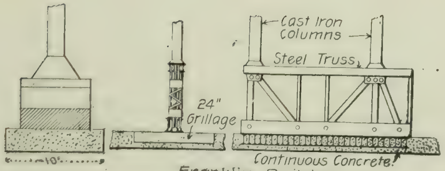
Gillender Building Foundations



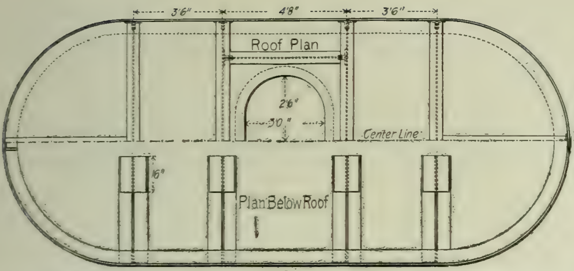
Detail of Broad Street Area Wall
Commercial Cable Building

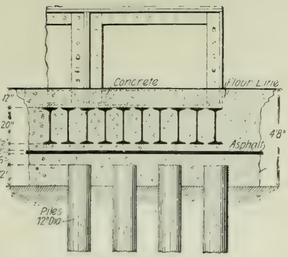


World Building

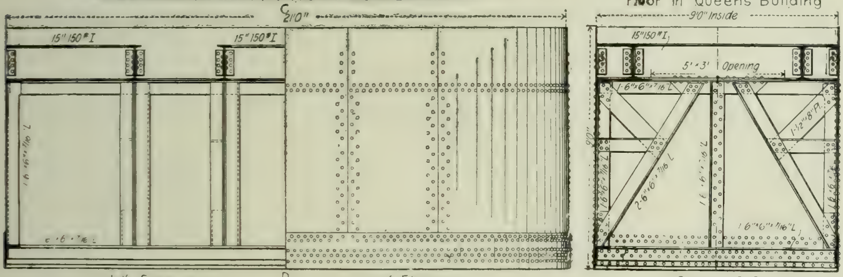


Franklin Building





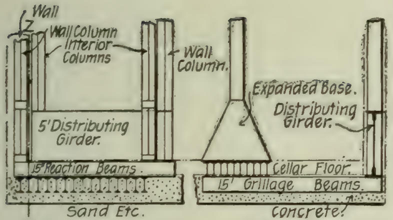
Section Through Foundation Floor in Queens Building



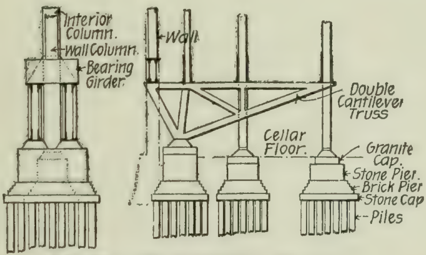
Half Section D Half-Elevation

Caisson in Standard Building

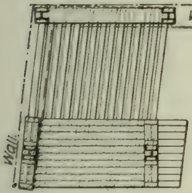
Section C D



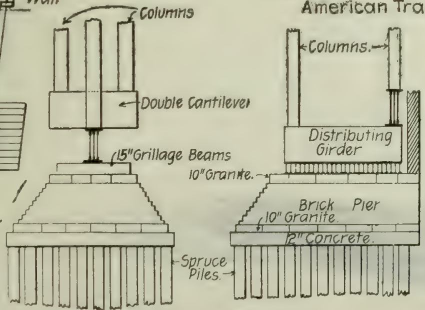
Elevation. Section Z-Z



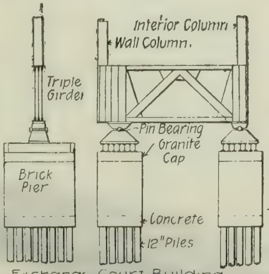
American Tract Society Building



Plan Wilkes Building



Park-Row Building Foundations



Exchange Court Building

THE GREAT HYDRAULIC LABORATORY AT CORNELL.

BY HARLEY S. GIBBS, '98.

Though no official report upon the construction of this laboratory has yet been made public by the engineers, sufficient data has been obtained to give the reader a general outline of the important features of its construction, its purposes, and some of the objects which its promoters by its means hope to accomplish. A large part of what is found in the following pages was taken directly from a statement made by Prof. E. A. Fuertes before the American Society of Civil Engineers at their Annual Convention in June, 1897.

Besides the hydraulic laboratory itself and all that appertains to it, there are two other important features in connection with this work; namely, the water supply of Cornell University, and the supply of water power to the turbines and power plant located some distance below the laboratory, from which the University will obtain under better conditions than at present its light, and motive power.

The location of the laboratory is a most fortunate one; indeed it is estimated that if its natural conditions were not so favorable, $3,000,000 could not provide the resources that nature offers almost without price, resources which are indeed priceless. The laboratory is situated mainly on the south bank of Fall Creek in the vicinity of Triphammer Falls where the banks are precipitous. A view of the site before work began may be seen on Plate I. This stream has a minimum delivery of 12 cu. ft. per second and a maximum delivery of nearly 5,000 cu. ft. per second, and from 2,000 to 3,000 cu. ft. per second can be relied on during the best seasons for work.

A dam across the stream has been built to store from 50,000,000 to 70,000,000 gallons of water.

The old Beebe dam, now dispensed with, which had for some time existed at this point, was used for storing water to be pumped into the University reservoir for the supply of the campus. The new dam is built 15 feet below the old one and rises 10 feet above it, causing the water to spread out behind it into a lake covering an area of 23 acres, which is supplied by a watershed embracing 117 sq. miles.

The construction of this dam is shown in Plates II and III. Though built in the form of an arc of a circle, it acts as a gravity dam and not as an arch. It is built entirely of concrete and together with the canal walls and gatehouses on the south end forms practically a monolith. It is 30 ft. high at the deepest section and is anchored securely into the bed rock of the stream and the ledge on the north side.

The excavations in the bed rock and the ledge, together with the old dam are shown, in Plate II, while the methods of laying the concrete on the moulds is shown in III. The steps shown on the downstream face are designed to break the force of the water during heavy floods and so prevent undermining of the foundations. As an additional measure of safety, the narrow space between the former Beebe dam, which is left standing, and the new dam, is rammed with puddled clay.

The rock promontory north of the dam has been stripped of trees and soil down to the rock ledge and levelled one foot below the upper surface of the dam. This area forms the spillway, and, during the major portion of the year, will carry the natural flow of the stream in a thin widespreading sheet covering its entire area and discharging into the narrow defile below the dam and above the fall. The slope on the north end of the spillway has been paved to protect the banks from floods, which may rise to depths exceeding four feet over this area and three feet over the crest of the dam. Provision has been made in construction for increasing the storage capacity, by adding flash boards over the dam and spillway.

At the south of this dam, a canal blasted out of the rock is provided with gates, and extends some 500 feet along the bank of the stream. It is 16 feet wide and able to carry a depth of 10 feet of water, which can be delivered into it under 20 feet of head. This is constructed of concrete, backed with asphaltic water-proofing to insure against leakage, and there is an underchannel for the thorough subdrainage of the surrounding rock strata. Between the canal and dam is a concrete wall which extends up stream and separates the water in the reservoir from the canal. This is shown in Plate IV in course of construction, and later in Plate V, which also shows the canal nearly completed. Within this wall is a tunnel (see Plate IV) to carry the water received from the wier upon a movable cradle discharging into it until the conditions of flow over the wier have become uniform, when the cradle is taken away and the discharge takes place within the prism of the canal. A very light and artistic concrete footway, not shown, spans the canal from the south bank to the gate house on a level with the top of this wall.

Behind the entrance gates of the canal, baffle boards and other contrivances control desirable conditions for the entrance of the water into the canal. The wier with which the canal is provided can be calibrated and replaced under identical conditions, using the water-tight canal to measure the volumes delivered. Within the walls, special contrivances to be read by filar micrometers are placed at short intervals from each other to give the height of the water along the canal with great accuracy. The canal can be emptied at pleasure, either in the shape of a waterfall about 80 feet high or into a steel pipe 72 inches in diameter and over 70 feet in height, which pierces vertically under it, the roof of a building 24 feet by 50 feet, in which can be conducted many experiments better performed under shelter. The elbow connecting this vertical standpipe with the branch from the canal is shown in Plate VI, while Plate VII shows the pipe in position and the site of the laboratory building at its base and under the cliff. By the side of the canal and parallel to it, a steel pipe 3 feet in diameter can deliver its water at will either into the 6 foot steel pipe, the canal, or directly into another building, to supply water pumps and dynamos for lighting and water supply purposes, under 80 feet of head. The end of this 3-foot pipe, with its branch to the canal and standpipe, are shown in Plate VII. This is the pipe which runs from the gatehouse at the south end of the dam, and which is to be continued on down stream from the end shown. Under normal conditions of usages the supply to the standpipe will be drawn entirely from the 3-foot pipe.

Halfway down the 6-foot steel pipe, a turret like structure, makes convenient the conducting of experiments to determine coefficients of efflux in thin plates, all forms of orifices, tubes, nozzles, valves, cocks, etc., up to 4 inches in diameter, and under different, constant and variable heads. In the building below, on the floor of which the standpipe rests, special castings and reducers can be attached to the steel pipe, capable of taking pipes and valves from 6 inches to 4 feet in diameter. Along the bottom of the creek there is room for experimenting upon these larger pipes for straight lengths up to 2,000 feet, and upon indefinite lengths beyond this distance, with both vertical and horizontal curves. A number of Venturi meters, which do not use electric counters, are provided for experiments of this nature. Provision is made for connecting the large standpipe directly with the University reservoir on the campus, thus giving a total head of 200 feet for special experiments.

Upon the walls of the canal itself, two rails support a truck provided with a small dynamo that can be adjusted to run the truck at suitable velocities. A chronograph marking seconds is made to register automatically the travel of the truck at every ten feet of its run and, therefore, its position and velocity can be determined with any desirable degree of accuracy. The water of the canal can be so regulated as to be either perfectly quiescent, or its velocity can be controlled by the amounts allowed to enter the upper wiers of the canal and the relative positions of the down-stream wiers. The height and the conditions of the position of all the wiers can be regulated with the greatest nicety.

It might be said here that the natural beauty of the gorge in the vicinity of Triphammer Falls is to be restored so far as possible at the completion of the work—the new structures being designed to harmonize with their surroundings.

While it would be impossible at present to detail all the experiments that experience with this laboratory may develop, the following programme is being planned :

1. Studies upon the dragging and suspending power of water at various stages of its saturation with sediment.

2. The effect of transverse, longitudinal, and submerged dams, under standard conditions, which may be modified at will by disturbing influences covering any variety of complications.

3. The determination of corrections to be made in the beds of streams to give them the most stable longitudinal profile.

4. Studies upon the conditions of such rivers as build their minor beds above their major beds. Major Leach's recent paper upon "What the Mississippi is, and what it needs," offers numerous suggestions of the usefulness of this laboratory.

5. Study of littoral cordon formation, and of channels, bars, and deltas, and of the disposition of sediment from rivers entering into quiescent water, and against high tides.

6. Study of the conditions affecting the length of tangents and degree of curvature in natural and constrained water courses, looking to secure permanence of channels and depths of water.

7. Studies upon the delivery and conditions of the water-shed of the stream and its tributaries which feed the canal, in reference to the amount and kinds of suspended matter by floods, the inter-relations of the deliveries of the tributary floods, and such studies as may prove useful for determining the co-efficients of flood volume, lengths of dams and spillways, and heights of floods over them, so as to perfect the formulas for the deliveries of water-sheds, if this be possible. The water shed of this canal, covering 117 square miles, will be most carefully surveyed topographically and geologically.

8. With this canal, experiments can also be made upon the rating of current meters, and general studies upon the motion of water in open channels, in pipes, and over wiers, under variable conditions of velocity, materials of the bed, conditions of surface, contractions, and heads.

9. The determination of the resistance to the motion of boats in canals in reference to their respective cross-sections, effect of waves, etc.

10. Experiments on water-jets, forms of water-wheel buckets, ratios of areas and forms of propellors, including water jet propulsion ; which subjects, by themselves, will give rise to a large number of investigations.

11. Experiments on the construction and efficiency of water and other motors, including water meters.

12. Effects of form and condition of the surface of the vessels upon their speed and motive power required.

13. The uses of this laboratory are not restricted simply to questions strictly classified as of hydraulic importance. For example, on the sanitary side, the relations that should exist between the grade of a sewer, its size, and the volume of flush water required to produce a given effect, are almost entirely unknown.

The work of S. H. Adams just published, does not supply the needs of sewer designers with needed generality. The equipment of this laboratory can settle the conditions required to determine how far down grade a definite volume of flush will be effective, under the varying conditions of this problem.

14. The engineer who is to supply drinking water to cities and makes it more or less stagnant behind huge dams and impounding and distributing reservoirs, has had, hitherto, no definite knowledge of the best methods for fostering and preserving a standard of purity in his water supplies. Since impure water is the cause of innumerable deaths, the opportunity that the laboratory offers for many important studies will be easily appreciated.

15. This very much enlarged laboratory can now continue, under the best auspices, investigations under way for some time past based upon the universal theory that all living beings derive their sustenance, at least in part, from other living beings ; and that certain conditions of temperature, light, and plant life, either arrest or foster their growth. An effort is being made to find out how to populate our drinking water with forms of life, capable of destroying dangerous organisms, and yet strong enough in their struggle for life to survive in sufficient numbers for this beneficial purpose under protective conditions, as yet difficult to control. The hydraulic laboratory lends itself admirably to studies of this nature.

From the preceding description of the numerous facilities of this laboratory, it may be readily seen that it is superior to anything built hitherto in any country. It will be open to the inspection, and under some conditions, use, of any engineer desiring to conduct such experiments as he may need in the incidents of his practice, or who may make inquiries or furnish data upon experiments of importance that he may desire to have made. The advantages for performing experiments and investigations upon such a large scale as this laboratory affords, have never before been available, and it is expected that light will be thrown upon many unsolved problems of far reaching importance, hitherto impossible of solution for lack of the proper facilities.

The only object for which this laboratory has been created is, "To Foster the Progress of Hydraulic Science." Its construction is now nearly complete.

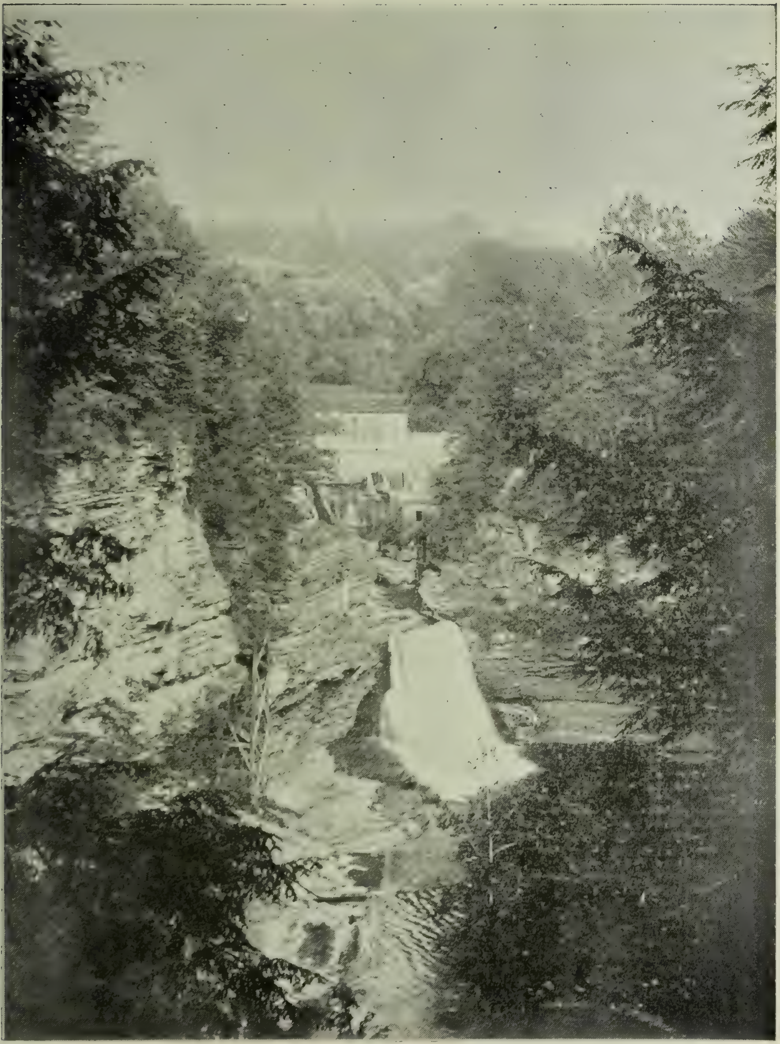


PLATE I.—Triphammer Falls ; Beebe Dam ; Pump House.
Site of Hydraulic Laboratory. Looking East.

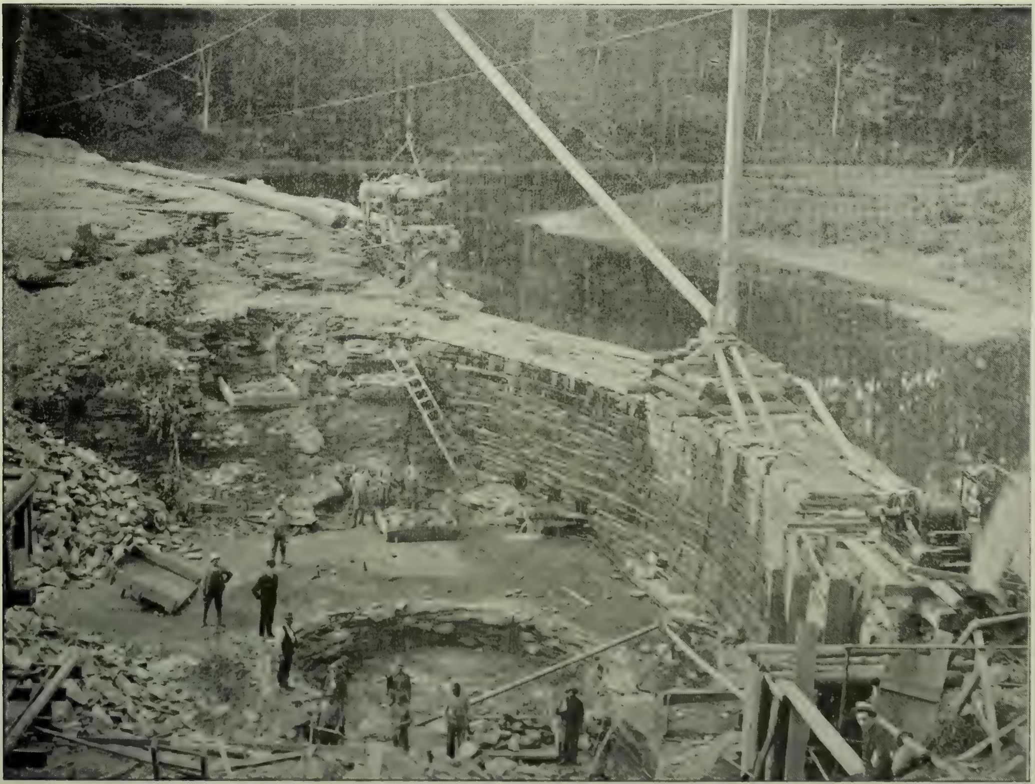


Plate II.—Excavations for New Dam ; Old Dam. Looking North.

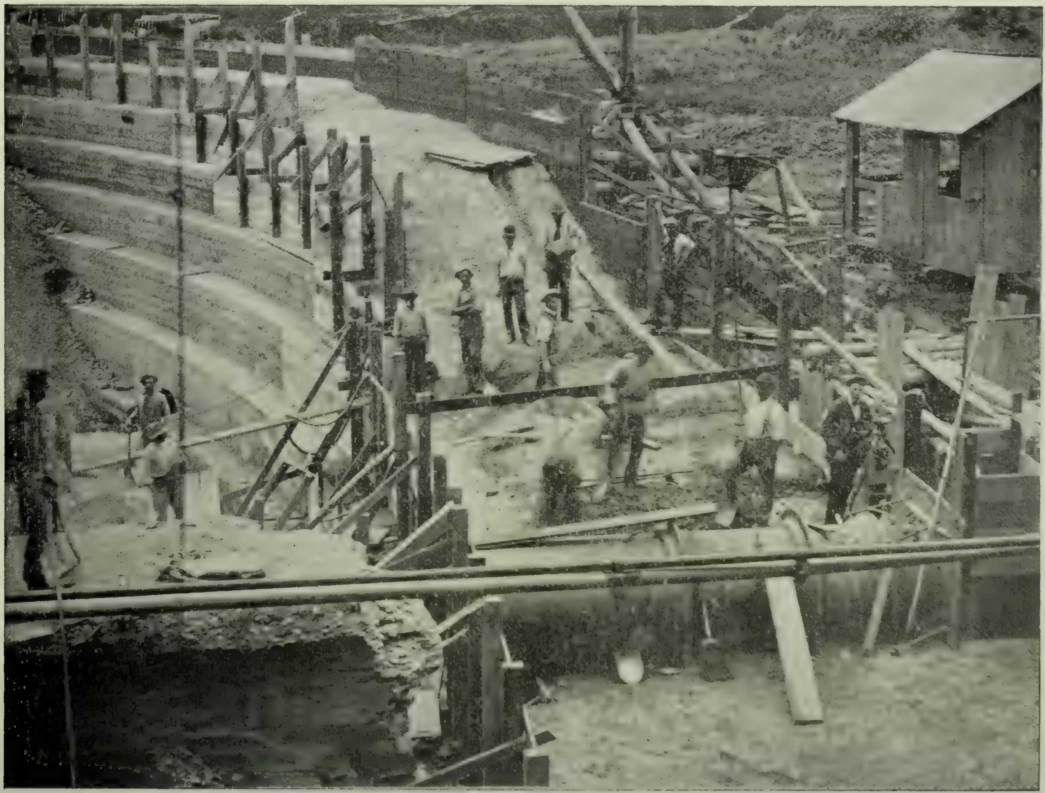


PLATE III.—Construction of Dam. Looking Northeast.

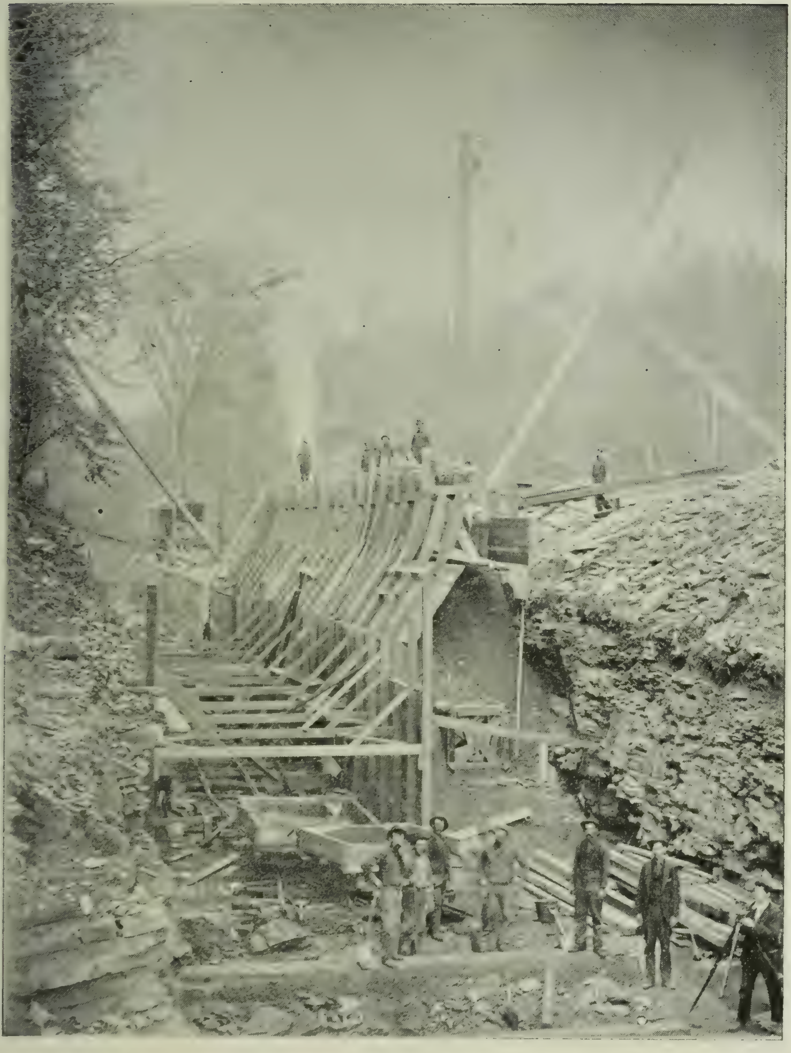


PLATE IV.—View of Right Retaining Wall of Canal, Showing Waste Weir Tunnel below Head Bay.

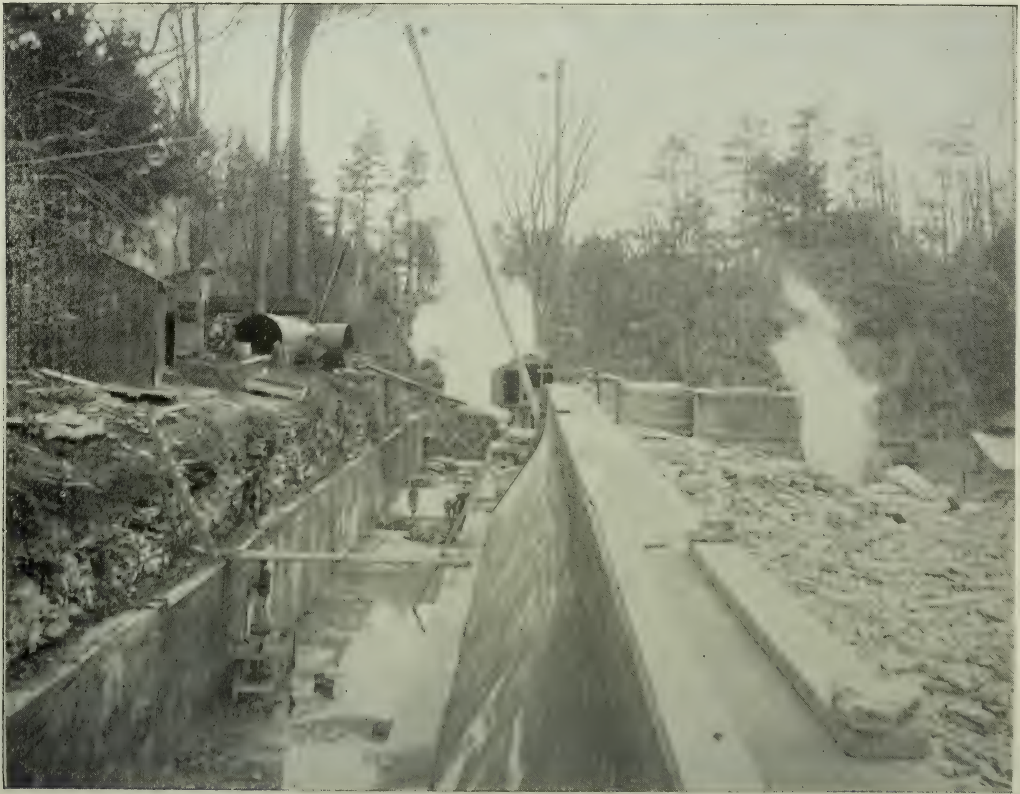


PLATE V.—View of Canal. Looking West.

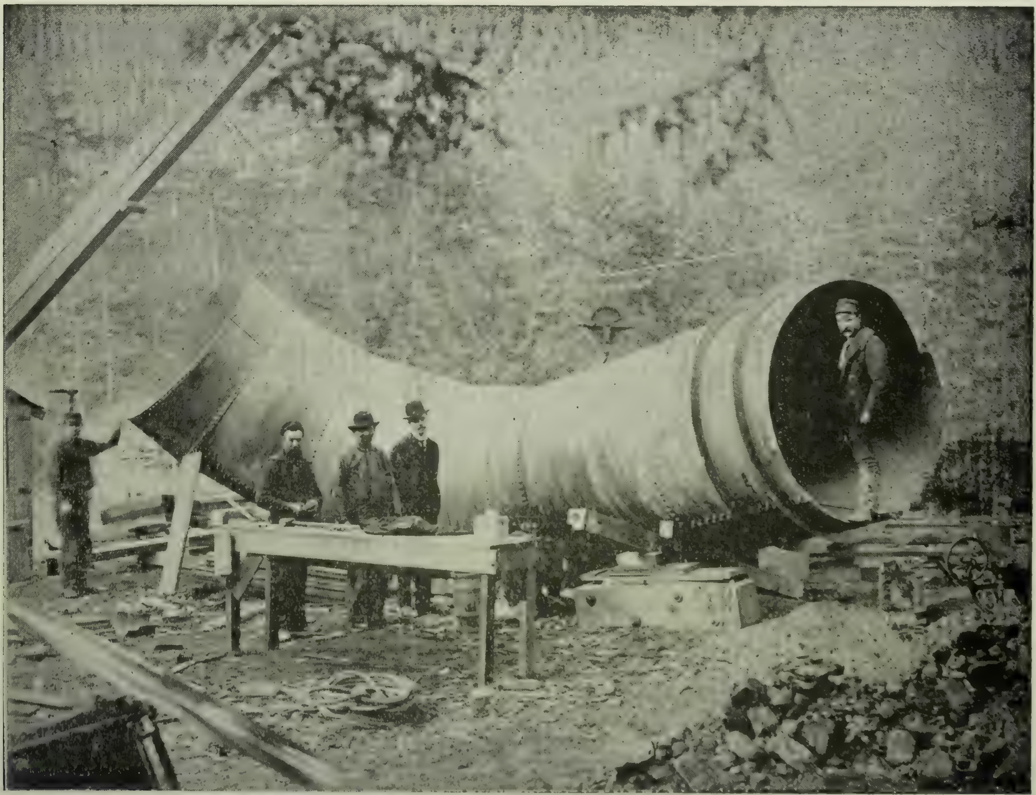


PLATE VI.—View of Elbow in Steel Pipe which Pierces Roof of Laboratory.

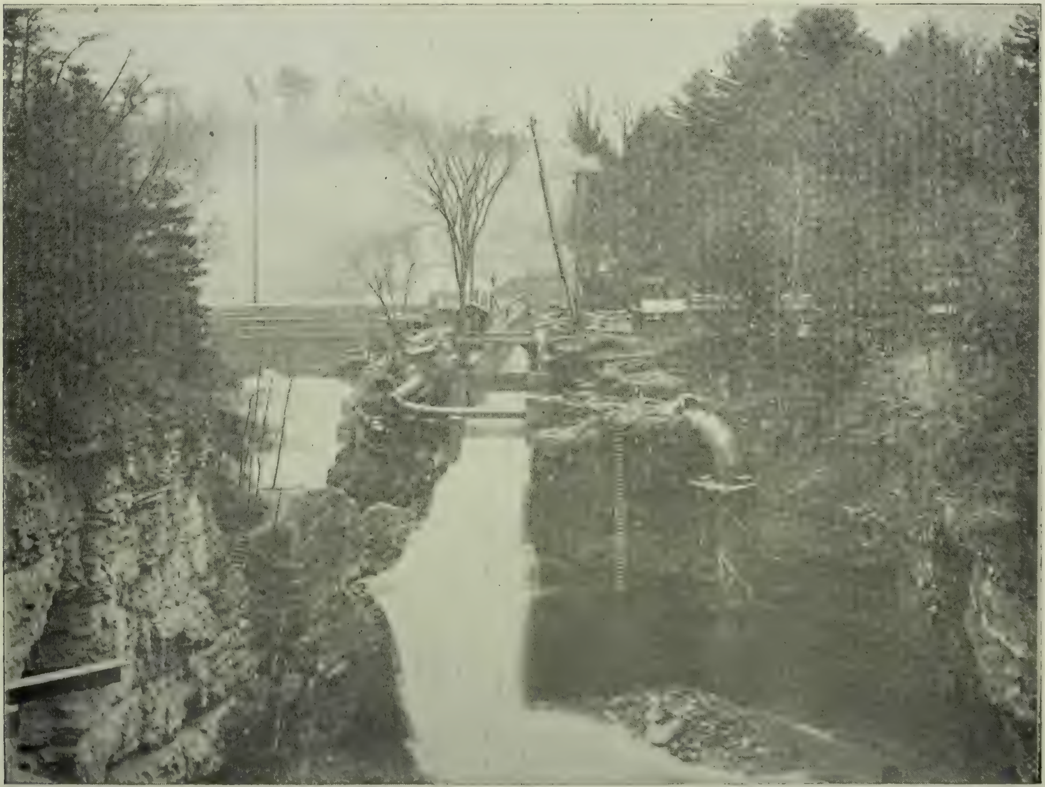


PLATE VII.—Site of Hydraulic Laboratory, Showing New Dam, Canal with Gates, Standpipe and Triphammer Falls. Looking East.

DESCRIPTION OF A TOPOGRAPHICAL SURVEY OF A FOUR-THOUSAND-ACRE TRACT.

EDWIN DURYEA, JR., C. E

In January, 1891, the author was engaged to make a careful topographical and boundary survey of a tract of about 4,000 acres at Tolleston, Lake County, Indiana, on the extreme southern shore of Lake Michigan. This land had just been acquired by Armour, Swift & Morris, the Chicago meat-packers, with the intention—should they be unable to come to an agreement with the Union Stock Yard Company, of Chicago, in a dispute about charges then pending—of transferring their immense packing-house industries thither.

The establishment of an industrial town was contemplated, to comprise, besides packing-houses, stock-yards, and homes for all employees, such features as a land-locked harbor connected by a short canal with the lake, a large dock-frontage on same, extensive railroad terminals, and miscellaneous manufactories, such as tanneries, furniture factories, etc.

The owners engaged the firm of Washburn, Shaler & Washburn, Civil Engineers and Contractors, to make the survey, and the work was done for the firm under the direction of the author.

It was apparent that, in view of the varying uses for which the tract was intended, and to so dispose and arrange the many industries as to prevent interference and to make the design a harmonious whole, very accurate and detailed maps would be necessary. It was also evident that such maps would be of great use, not only in locating but also in designing all improvements, and especially in giving data, without additional surveys, for accurate preliminary estimates of cost.

The tract to be surveyed and mapped was composed of eight fractional sections, with a total area of 3944 17 acres. It had a frontage of four miles on Lake Michigan and a depth back from lake—except for the west mile, only one mile deep—of about two miles. It was traversed from east to west by the Grand Calumet River and by the main lines of the Baltimore & Ohio Railroad and of the Lake Shore & Michigan Southern Railroad ; and the main lines of the Michigan Central Railroad and of the Pittsburg, Fort Wayne & Chicago Railroad—the Pennsylvania System—passed within a few hundred feet of its southern boundary

The soil, except the muck in river-valley and sloughs, is a pure, fine lake sand, its surface formation being in general that of successive ridges parallel to lake shore and to river;—in many places, however, these ridges were so broken—apparently by wind—that the general formation was hardly perceptible. The extreme east of tract north of river and a narrow belt along lake beach were covered with jack-pines and some small junipers; and all the remainder of tract—except the river valley and lake beach, which were bare of trees—by second-growth red-oak scrub with occasional older trees and a very few scattered pines. The thickness of the growth varied considerably, but might in general be called medium; in no place, even for short distances or in winter, could lines be run with transit or level without considerable chopping. The timber was of little or no value except for firewood, but seemed to protect the land, otherwise nearly bare of vegetation, from the action of winds.

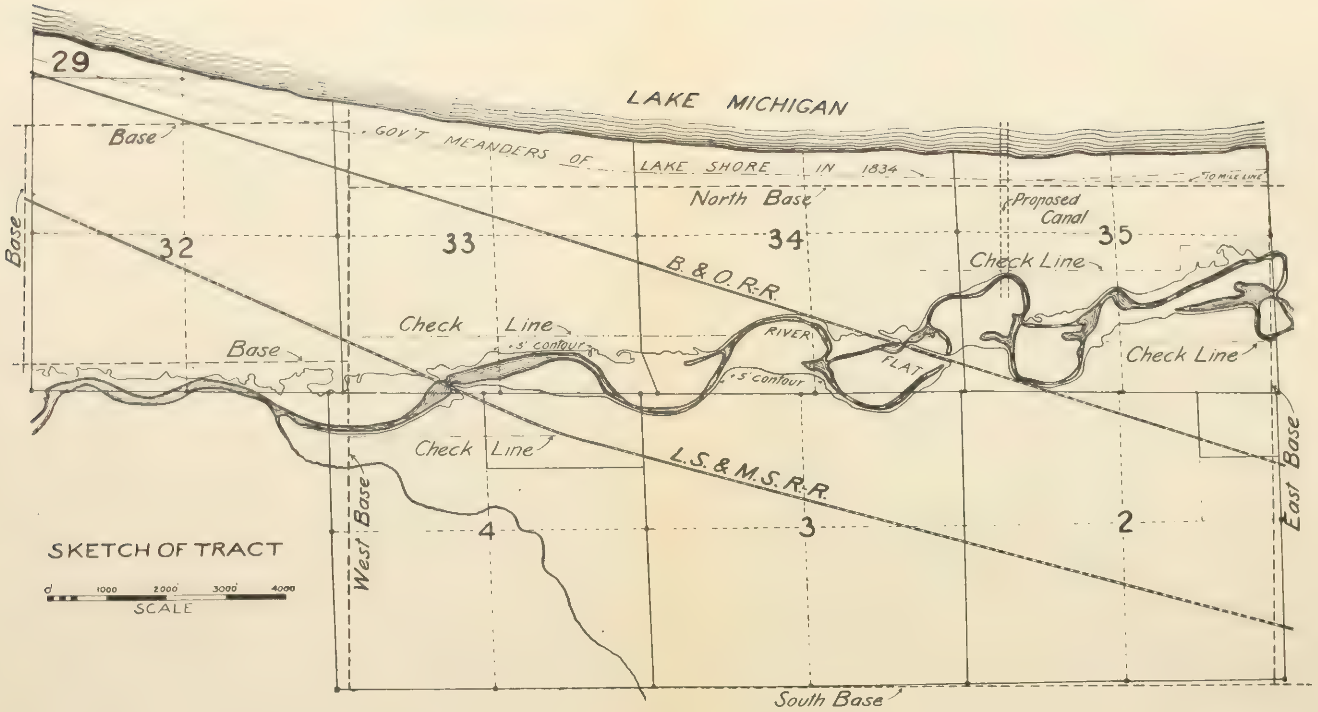
The only other natural features were: the presence in Sections 2 and 32 of numerous long, narrow, and deep marshy sloughs parallel to the river between the ridges; a small and tortuous stream flowing into the river from the south; and the extremely broken character of the surface, which might be said to be composed of chains of ridge-like mounds having about the same general elevation, with a height above valleys of say from 10 to 25 feet. The whole tract—leaving out river-valley—would probably grade level to an elevation of about 17 feet above lake-level, while the extreme elevation—on one mound only, in north-east portion, with no other elevation nearly so high—was 78 ft. above lake-level.

The tract was almost entirely destitute of improvements; the only exceptions being the two R. R's with their right-of way fences, a small shanty used as a B. & O. R. R. telegraph office, a group of several rough buildings near the lake shore used by a hunting club, and one small and almost valueless squatter's shanty near the river.

A day spent in examining the tract gave the following conclusions:

1st. That the maps, to be of any value at all for the purpose in view, must be in great detail. For while with a country having well-defined features and regular slopes and contours, a map giving the salient points only may be of great value, which value can be increased in direct proportion to the time spent in acquiring additional details; in the case of the tract in question, with no well-defined lines of structure, and for the uses intended—city improvements such as sewers, grading streets, etc., in addition to the R. R. and manufacturing features—no maps except such as gave minute topography would be of any use or other than misleading.

2nd. That because of the wooded character of the land and its


SKETCH OF TRACT

general sameness of elevation, no use could be made of such quick methods of surveying as triangulation, the stadia, or the plane table—but that all lines must be run out with transit and chain.

3rd. That this transit-work could be done much quicker and cheaper in winter than in summer, both because of the comparative absence of leaves and because of the ice on river, sloughs, and marshes.

The essential points it was decided to embrace in surveys and to embody in the maps were as follows:

1st. Contour-lines 5 feet apart over whole tract, with elevations of crests and hollows in addition.

2nd. Shore-lines and surface-elevations of lake, river, streams and sloughs.

3rd. Depths of water in river and sloughs and soundings to determine depth and character of hard bottom wherever surface was soft and unfit for foundations; and a sample of seepage or slough water from each section for analysis.

4th. Notes on character and value of the timber and of any other products.

5th. Locations of R. R's and of their rights-of-way; and of all buildings or other improvements of any kind.

6th. A complete and accurate survey of all government section lines, this to include the re-establishment of all lost Government Survey-Corners and the monumenting of all corners—the survey and perpetuation to be made legal as provided for in "Revised Statutes of Indiana, 1881."

7th. The establishment of accurate bench marks on all Corner-monuments and at many other convenient points; and the perpetuation for future use of many of the transit-points accurately located during the survey.

All the information obtained was to be embodied in the following maps: 1st. a set of 8 contour maps on scale of 1″=100′, each map to be 58″ × 60″ and to be devoted to a single section; 2nd, a single contour map of whole tract on scale of 1″=400′, to be reduced from above set and to be 58″×84″; 3rd, a map of whole tract showing legal survey only, of same scale and size as last and showing government section-lines as determined, with distances, bearings, etc., but no topography; and 4th, a map for filing in county surveyor's Office, very similar to last but on scale of 1″=800′ and containing descriptions of each monument set and of all bearing-trees or references for same.

The article to this point may be regarded as almost entirely a statement of the problem to be solved: showing 1st, the nature of tract to be surveyed; 2nd, the information to be obtained by the survey; 3rd, a list of the maps by means of which this information was to be recorded so as to be available for future use.

The remainder of the article will be divided into four parts :

1st. A Journal of the Field-Work ; 2nd, A Journal of the Mapping; 3rd, Methods used and Results attained ; and 4th, Cost of the Surveys and Maps.

As the main object of writing this article is to describe the methods used and their results, the writer will devote most of the space to that heading ; and in doing so will not adhere closely to the lines of this special survey, but will include briefly the mention of other methods, or the application of these methods to other surveys.

JOURNAL OF THE FIELD-WORK.

On January 2nd, 1891, the writer visited the tract with Mr Frank S. Washburn, of Washburn, Shaler & Washburn, going over nearly all parts of the tract on foot. He then returned to Chicago, where the time until the 7th was spent in getting together a surveying outfit, arranging for a party, and making general preparations for the work.

On January 8th he went over the tract with the county surveyor in order to learn from him the locations of the government survey corners and their comparative reliability.

The actual work of the survey began on January 10th, when the writer located at Tolleston with one man, a leveler, hired an ordinary laborer for a temporary rodman, and began the work of transferring levels from a U. S. Engineer's bench-mark at Hammond, Indiana, 10 miles distant.

Until January 22nd, while waiting for the filling up of the party, the work was confined to running trial section lines, testing the reliability of the government survey corners as then marked, etc ; and on the 23rd the topographical survey itself was begun, with the preparation of a straight base-line three miles long on the south edge of the tract.

From last date until February 10th the time was spent in preparing and measuring 9.57 miles of base-lines forming a quadrilateral—and practically a rectangle—around the east three miles of tract ; in meandering the Grand Calumet River on the ice and sounding river for depths of water and of hard bottom ; in completing connection with the government bench mark at Hammond and with another at Hegawisch, three miles further away ; in establishing bench-marks at convenient points in the tract ; etc., etc.

On February 10th the work of collecting information for topography and contours began. It had been decided that levels for contours should be taken on and at right angles to parallel north and south lines, spaced from 100 feet to 300 feet apart and connected by the base-lines; and on above date the transit work on these lines was begun. This work was continued till its completion on March 17th, two transit parties being employed February 11th to March 5th and one the remainder of the time.

On February 14th the work of taking levels for contours was begun with one level party and continued till its completion on March 28th, three level parties being worked February 25th to March 22nd and one at other times.

The sloughs on tract—mostly in sections 2, 3 and 32—were sounded for hard bottom and depth of water March 3–9, advantage being taken of the ice; and the railroads were traversed and connected with the east and west bases March 5–7. On the completion of the transit lines for contour levels on March 17th, the transit party began the preparation of the base-lines on the south and west sides of section 32—the only bases not already completed—and completed them March 19th.

The legal or boundary-line survey, dropped on January 22nd, was resumed on March 19th and carried forward with one transit party till May 28th; the field-force was then reduced to an instrument man and one axe man, and the boundary-line survey entirely completed by them on June 8th. Many miscellaneous surveys were made by this transit party after March 19th, including checking the alignment of the bases and perpetuating them by permanent hubs, various lines run to check the accuracy of the chaining on the transit lines for contours, remeasurements of these lines to locate errors, etc., etc.

From completion of levels for contours on March 28th, a leveler and one man were employed almost constantly to May 24th in sketching shores of sloughs, looking up field errors in levels and stationing, discovered during the office-platting, taking topography of railroads and buildings, taking samples of water for testing, and a multitude of small miscellaneous jobs.

A true meridian was established by observations on Polaris on the nights of April 27th and 30th:—and on June 8th a standard 100 feet was laid off between iron monuments,—as represented by the tape on which the entire survey depended, at 62° Fahr., under strain of 12 pounds, and supported only at the ends or 100 feet apart.

Levels were transferred to the iron monuments marking government survey corners June 8th to 18th, and the entire field-work of survey completed on latter date.

JOURNAL OF THE MAPPING.

All lines and levels were platted from field-notes on the 100 foot scale maps, the contours deduced from them, and the 400 foot scale map made from these maps by reduction.

The mapping began March 2nd, 1891, and until April 5th, consisted entirely of platting field-notes on 100 foot scale maps; one draftsman being worked until March 5th, and two afterward. The work of deducing contours from the platted elevations began on April 6th with the arrival of a third draftsman, and a fourth was added on the 11th.

On April 16th the reduction of contours to 400 foot scale map was begun, proportional dividers being used until the 22nd, when a pantograph was was put in use; but the draftsman using the pantograph leaving for other work on the 25th lowered the drafting-force to three men and suspended the reduction for a time. On May 6th the drafting-force was increased to five men, and the reduction by pantograph resumed and completed on 19th.

The inking of the countours on the 400 foot scale map was begun on May 9th and completed on the 20th, most of the inking being done in extra hours, before 7 a. m. and after 6 p. m., so as not to delay the reducing.

On May 23rd, after two days spent in adding soundings, land-lines, etc , the 400 foot scale map, with preliminary estimates of Canal, Drainage of sloughs, etc., was laid before the owners of tract in Chicago; it was then returned to Tolleston, and after a few days spent in adding information in pencil, was turned over to a skillful draftsman in Chicago for the addition of title, border, and topography.

The platting of all field-notes—mainly levels—on the 100 foot scale maps having been completed on May 12th and all contours being penciled on 14th, on the 15th the drafting-force was reduced to three men. On the 13th the inking of the contours was begun and completed by two men by the 18th; and the contours were numbered in ink by one man (with interruptions) from the 18th to the 28th.

The drafting-force had been reduced to two on the 20th and to a single man on the 23rd; and from May 28th till his discharge on June 4th, this draftsman was engaged on an 800 foot scale map of legal survey for filing in County Surveyor's Office.

From June 6th to July 24th the eight 100 foot scale maps were completed by adding in ink selected elevations of crests and hollows. The maps were at the same time thoroughly checked and examined for errors and the errors corrected; the writer during this time spending 20 days on these maps and a draftsman—previously an instrument man on field work—36 days.

During this same period titles were put on these maps by the draftsman in Chicago; corrections made in the 400 foot scale contour-map to correspond with those mentioned above ; a map of the legal survey made on 400 foot scale ; and the maps carefully cleaned. On July 24th, the maps were turned over to Washburn, Shaler & Washburn, completed.

The writer is aware that some apology is needed for presenting the two journals in such detail. He has done so in the hope that—especially to undergraduates and the younger graduates—the matter will be worth the three pages it occupies as showing the natural sequence of the different parts of the survey, the building up and reduction of the party, and the effort at all times to adapt the party to the work at hand, or, when not practicable to do that, to use the available party on such work as would most tend to hurry forward the survey. The journals should also help to show the application of the methods used—which might be common to many surveys—to this special one.

Some general features affecting the work require mention in connection with the journals.

As the most important, it was at no time during the formation or increase of the party practicable to get as many, or—in many cases—such quality men as were wanted. This was largely due to the short duration of the positions offered ;—and was especially true of the drafting-force, which usually more nearly resembled a school, with the writer as teacher, than an Engineer's office.

In Tolleston, a small German village of perhaps 200 people, it was found very difficult to get office-room or even hiring-accommodations for the party—which at its maximum, including the drafting-force, numbered nearly thirty men. We were almost forced to seek accommodations at a larger town about ten miles distant on a railroad, but arrangements were finally made at Tolleston, the office being divided between rooms secured in two houses about 500 feet apart.

Lastly, it being practically impossible to get over the tract with a wagon, teams were used very little. While doing transit and level work for contours in sections 34 and 35, the party so employed were boarded, though unwillingly, by the hunting club on section 34.

The hours of work were usually from 7 a. m. to 6 p. m., these being the hours at which the parties departed from and arrived at Tolleston.

METHODS USED.

The field-work methods will be described first and will be sub-divided into those used in the topographical survey and those used in the boundary line or legal survey.

Topographical Survey: – Under the plan of survey adopted the first step was to surround the tract with rectangles formed of accurate base-lines, in locations shown by heavy dotted lines on map.

It being desired to use a system of rectangular co-ordinates both in the field-work and the mapping, the south base line (along south line of sections 4, 3 and 2) was selected as the axis or zero of north co-ordinates, and an imaginary line at right angles to it through the southwest corner of section 4 as the zero of east and west co ordinates.

The bases were prepared as follows:—Stakes 2″ to 3″ in diameter were driven well into the ground at distances 100 feet apart, measured on the natural slope of the ground, the tops left about a foot above ground, and tack centers carefully set in each stake.

The transits available for use were not suitable for such work, the best being a railroad Gurley which had been in use for a long time and the others being practically worn out for accurate work. The only safe way to use them in running accurately straight lines was to use two hubs as back-sights as a check, then set a hub for next transit-point by double or reversed sights, and line in station-stakes by running toward this hub as a fore-sight. The stakes were "stationed" consecutively by their co-ordinate numbers—those on the south base, three miles long, being numbered from 1 E. (East) to 159 E. Intermediate unnumbered stakes were placed wherever the ground was convex between the stations, to keep the tape from touching the ground; special care was also taken to remove from between the stakes all brush and grass which could touch the tape.

The seven base lines, of 12.30 miles total length, were prepared in about 14 day's work of one transit party or at the rate of 0.88 miles per day; and the cost of preparation (see table of cost), was $16.90 per mile. The party was composed of a transit-man and from 4 to 6 men—two chain-men, a back-flag, and the others axemen either clearing the line or making and driving stakes.

The cost and time were much increased by the cold weather, the sand—especially on bases near the lake—being so frozen that it was very often necessary to cut holes with axes in which to set the stakes.

Elevations were taken on the tops of all these stakes and gave corrections which reduced the inclined distances as measured by tape to the horizontal distances.

The tape measurements were made with a 100 foot steel band-chain, graduated every 5 feet and end foot to 10ths of a foot; this is the cheapest form of steel tape and costs about $4 each. In measuring, the tape was suspended clear between stakes for whole lengths and put under a tenson of 12 lbs. The entire 100 feet of tape was used and a + or −

correction, usually less than an inch, was read by a common rule to $\frac{1}{80}$th of an inch to obtain the exact distance between tacks; all distances were measured to the north or west edge of tacks, unless tacks were especially marked with a center line.

The temperature was frequently taken and all measurements afterwards reduced to + 62° Fahr The thermometer used was the most common kind, costing only 25 cents; and the spring-balance used to give a uniform tension to the tape was the ordinary kind weighing to 24 lbs. and costing only 25 cents.

The bases were measured by a party of three men, one of them corresponding in rank to a transitman, and the work was done at the rate of out and back over from three to four miles per day. But two measurements were taken, one out and one back, and no care was used in choosing the weather, it being usually sunny and somewhat windy.

The accuracy of the results is shown by the following table:

Base.	Length.	Difference between the two corrected lengths.	Variation of each length from their mean.
South	15,931 feet.	0.45 feet.	1 : 71000
North	15,880 "	0.05 "	1 : 635000
East	8,873 "	0.07 "	1 : 323000
West	9,827 "	0.12 "	1 : 170000
South of Section 32	5,196 "	0.16 "	1 : 65000
North of Section 32	5,155 "	0.026 "	1 : 390000
West of Section 32	4,055 "	0.07 "	1 : 116000

The comparatively great error of south base was due to the measurements for two miles being made through sloughs, with the ice breaking and the tape dragging in the water; and with weather sunny and slight breeze—all the worst possible conditions as affecting temperature corrections The cause of the error in the south base of section 32 has been forgotten, if ever known. Much better results than some of above could easily have been obtained as explained below, but the bases were needed for use at once and the least accurate results obtained were, besides, fully as good as needed.

This method of measuring base lines is similar to that used for base lines for large bridges, though much rougher. The additional precautions in bridge work are mainly in the choice of weather—a careful restriction of all final measurements to cloudy, quiet days, or to quiet days after sunset:—and exact care is taken in all ways with the measurements. In bridge triangulations it is more usual to get the stakes about 99½ feet apart and read the observed distance directly on the tape to thousandths of a foot—the tapes used being graduated to hundredths of a foot. The tape is also sometimes supported at two intermediate points by hooks or nails attached to stakes. In all cases, whether for the most accurate bridge work or for rougher uses, such as this paper describes, it is necessary to make two readings for each result entered in the field notes—after reading the tape once with great care and a certain knowledge that the tension and zero end of tape are properly attended to at the instant of reading, and that the tape is hanging perfectly clear and not twisted, the tension is removed and the readings repeated until the same result is twice obtained—when it and the temperature are recorded.

Great care must be taken not to bring a horizontal pull on the stakes;—to prevent this, the two men holding the tape in tension usually support the pull on an inclined stick running from the hand holding the tape to the ground. Four men are needed for making careful measurements by this method.

This method of measuring base lines was taught to the writer in 1884, on his first construction work, and is gradually superceding the more cumbrous methods and apparatus. It has been adopted by the Missouri River Commission, and the writer thinks by the Mississippi River Commission. Its practical success is probably due to three facts—(1) that a distance between two fixed points can be measured much more accurately than the same distance can be laid off, and with much less "personal equation"; (2) that the different measurements can be compared with each other not only as a whole, but also station by station, so that any disagreement can be easily located and checked; and (3) that the great length of tape compared to that of any rod-apparatus decreases the number of operations and of observations and, in consequence, the chance of error.

Some examples are given below of the results obtained in bridge triangulations:—

Cairo Bridge—

Ill. base, 3400′ long, 2 corrected measures differed 0′.005; Ken. base, 2400′ long, 2 corrected measures differed 0.006. The bases were about a mile apart and the computed length of Kentucky base differed 0′.02 from measured length.

Rulo Bridge—

The Nebraska base, about 1100′ long, was measured three times;—two of the corrected distances agreed exactly to a thousandth, the the third varied 0′.001. The measurement of the Missouri base is forgotten, also its agreement with computed length.

Keithsburg Bridge—

The bases were a half-mile apart and the computed length of Iowa base, 2400′ long, differed 0′.02 from its measured length. The other data are forgotten.

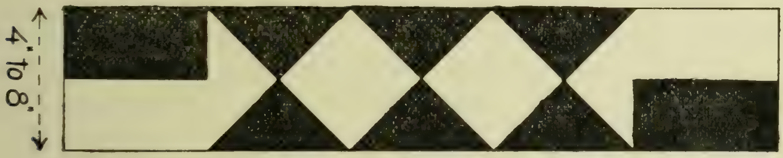
New East River Bridge—

A line 1391′ long measured three times, probable error of the mean was 0′.0034. Two measures of a line 814′ long differed 0′.003.

In the examples given, Rulo is the only one where the base was located on exceptionally good ground ;—in all other cases, they were located on rough or soft ground or in crowded city streets.

To conclude this digression on bridge triangulations, the writer believes that with the methods described the results can without special effort be kept within the following limits of error :—

Base lines, 2 measures to differ not more than $\frac{1}{500000}$ of length. Angles, each read by 3 sets of 5 repetitions, to close triangles within 3″ or more often 2″ :—the transits to be ordinary good 20″ instruments and the targets to be specially and very carefully painted as in sketch below, or similarly :



After measuring the bases around the tract, the next thing in logical order (though not done next in order of time) was the measurement of the angles which they made with each other. There were but 8 angles in all, 4 in main system and 4 in section 32 ;—in running these bases, all angles except the two closing angles (at northeast of section 35 and at northwest of section 32) were turned off as right angles in the ordinary way and by the inaccurate transits mentioned. These angles were all carefully measured afterward by repeating, usually taking 3 sets of 4 repetitions each, and thus getting a comparatively accurate measure of each angle, even with a poor transit. The final closing errors of the quadrilaterals were probably less due to inaccurate angular measurements than to slight kinks in the bases, or to the short foresights in some of the angular measurements ;—some of the foresights being within one or two hundred feet of the transit, because of the roughness of the ground.

The accuracy attained by the base systems is shown below :—

Main System :—The closing angular error was — 0′55½″. After adjusting angles so their sum was 360°, the co-ordinates of the northeast intersection were computed in two ways. The co-ordinates obtained by using the south base and the east base differed from those obtained by the use of the west base and the north base as follows :—

Difference between 2 values of north co-ordinates = 1'.19
" " 2 " " east " = 0'.03

The large difference in the north co ordinates is probably due to an error in the angular adjustment acting on the three mile north base, or to an unfound kink in this base.

Section 32 System :—The closing angular error was—1'47" ;—and the two sets of values obtained for the northwest intersection varied by 0'.17 in their northings and by 0'.25 in their westings

Before the base lines were all completed, the Grand Calumet River was meandered to take advantage of the firm ice ;—the meander lines were run in an ordinary way with transit and tape and were tied up to the west base, the east base and the west base of section 32. Immediately afterward the river was sounded through the ice for depth of water and of hard bottom, the soundings being located by the meander-line station.

It was realized before the survey was begun that the largest item of the field work of the topographical survey would consist in the transit lines which were to be used as a basis for the contour levels ;—and that any means tending to materially hasten this feature of the work would materially decrease both the time and cost of the survey. It was, therefore, decided to make the work on these transit lines as rough and as rapid as was considered prudent, and to confine all the precise work to the bases and to other easterly and westerly lines intersecting the roughly run tranits lines and affording checks on the correctness of their stationing.

Transit-lines for Contour levels.—A total length of 134 miles of these lines was run, 52.6 miles south and 81.4 miles north of river, all being parallel north and south lines Their main duty was to locate on the maps the ground elevations to be taken by the level parties ; but they also served to determine the lake shore and the edges of river-marsh and of sloughs.

A north and south direction was chosen for these lines because, the general direction of ridges being east and west, the lines would thus cross the contours more often at right angles and so give more information as to the shape of the ground.

These lines were run by setting up a transit on some station stake of north or south base, turning a right angle, and running a straight line north or south usually till it reached the lake or the river ; then by means of two right-angles and a measured off-set a parallel return-line was run, its connection with the base serving to check the parallelism of the two lines and to give a comparison between their stationings. Checks on the absolute error of stationings were obtained by noting the stations at which the lines intersected the railroads and a few other lines run for this purpose.

The transit-lines for contour levels were run much as railroad locations are run, but not so carefully—much less care taken to keep the chain level, etc Permanent hubs were left at every transit point and a numbered station-stake of lath at each 100′. Common 100′ surveyor's chains were used; 100′ band-chains, such as were used on the bases, were tried but broke so often because of the rapid work and the rough ground and stumps that their use was abandoned.

The extending of transit-lines from base to lake or river at one operation was departed from somewhat in that, lines crossing sloughs on sections 2, 3 and 32 were at first run only far enough to include all sloughs, every effort being made to finish all work (transit, level, soundings, etc.) on sloughs and river while ice lasted. The lines thus partly run were afterward extended to the river or the lake.

The usual size transit party on these lines was seven men;—comprising transit-man, head-chain, hind-chain, back-flag, man to drive and mark stakes, and two axmen. An experienced chainman was always used as head-chain, but all the other men under the transit-man were day-laborers picked up in the vicinity. The party was so organized as to get the greatest amount of work done—enough axmen provided to prevent undue delay to the transit work and chaining. The above party contained three axes, and occasionally an additional axman was used where the woods were unusually thick. The rate at which these lines were run is shown by the following table, which shows the line run per party-day—or per day actually worked per single party:—

Feb. 10–14;	aver. per party-day	= 2.33 miles	;—max. per party-day	= ———
Feb. 15–21;	" " "	= 2.65 "	;— " " "	= 3.16 miles
Feb. 22–28;	" " "	= 2.97 "	;— " " "	= 3.88 "
Mch. 1– 7;	" " "	= 3.20 "	;— " " "	= 4.17 "
Mch. 8–14;	" " "	= 3.84 "	;— " " "	= 4.56 "

At the last date but about five miles remained to do, scattered, and this was run in less than two days by one party

The entire 134.0 miles was run in 44.7 party-days, or at the average rate of 3.02 miles per party-day.

The most interesting feature of above table is the continuous increase as the work progressed in the miles run per day;—and the evident fact that had there been more of this work to do, the rate per day would have continued to increase for some time. This steady increase was due to several causes: largely to the common laborers in the party becoming used to their duties, and to discharging the poor men and retaining only the best;—all of the party except transit-man and head chain were generally Germans, slow to learn and to move, but steady and faithful workers. The transit man and head-chain of both parties were experienced and rapid workers.

The most effective cause in increasing the rapidity of this work, however, was the rivalry between the two transit parties simultaneously engaged on it;—each party having to report its own day's work every night and learning the extent of the other party's.

The increasing rates in the table were affected adversely, if at all, by the special ground surveyed, as the roughest ground in the tract was surveyed during the last three weeks.

The chaining on these lines fell short of the true distances as given by base lines by from 2 feet to 15 feet per mile, usually about 8 feet;—this was due mainly to not leveling chains with much care, all the ground being rough. The lines usually closed on their base (after running out and back, nearly two miles of line) within about two feet, both as to distance and parallelism;—thus showing that the chaining was uniform, the lines straight, and the angles right-angles—all to at least as great a degree as was needed.

The principal use of these lines being to give sufficient data to make possible a close location of their station-stakes on the 100′ scale maps, a close knowledge of their error at any point was necessary to make the corresponding correction in the platting. As already mentioned, these errors were determined by reading the station of each line at its intersection with the railroads and with four other lines which were run parallel with north and south bases in the locations shown on the map. There were also a few lines north of the river, run from north base, connected by triangulation with the corresponding lines south of river and run from the south base. All the transit-lines in section 32 were run from its north base and their errors determined by the railroads and by the south base of section 32, run after the completion of the transit lines.

These transit-lines were not usually extended beyond the edge of the river flat, which below the 3 foot contour was little better than a peat-marsh and impassible a large part of the year. The river itself had been previously located by offsets from a meander-line to the edge of the ice, the high banks were fully defined by the transit-lines, and the river-flat lying between was flat and featureless.

Levels for Contours.—It will be noticed that the leveling of these transit-lines was begun only four days later than the transit lines themselves. This was of course due mainly to a general effort to hurry the work, especially on that part of the tract containing sloughs; but this early start was very necessary for another reason, to give a sure index as to how close to each other the transit-lines should be run. Too great a distance apart made the level-work slow or inaccurate, or both—slow because of bushes, etc., delaying the sighting and inaccurate because of too great difficulty in locating the side-levels.

Throughout all the work an endeavor was made to have the level work closely succeed the transit work, using three level parties to two transit parties for this purpose :—by so doing not only was the work hastened, but the general supervision of the work made easier because of the parties working near each other ;—better data were also obtained as to distance apart at which to run transit lines.

In running the levels, elevations were taken on the transit-line at all station-stakes and at all very marked changes of slope, also at edges of slough, river, and lake ; side-levels were also taken, located by pacing (as were all levels taken on line between stations), for half the distance to adjoining lines. The intention was to have these side-levels taken only on tops of knolls, bottoms of hollows, or at important changes of slope ; but it was often hard to judge just where they were needed and the parties often fell into the routine of taking them at even distances from the line, as 25′, 50′, 75′, 100′, etc.

As a result of such routine, more elevations than necessary were often taken but all were platted and served as checks on each other ; at midway points between the transit-lines the levels were practically duplicated, one set from each direction, and if any discrepancy appeared when platted it was settled by looking up the error on the ground.

The side-levels were rendered quite difficult by leaves, the oaks growing quite low and scrubby and holding many of their leaves over winter. If taken in summer, the leaves would probably have made the level and transit-work one third slower and more expensive. Very early in the work it was found that two rod-men could be used by each leveler with advantage, about one-quarter more line being run per day than with one rod-man and at the same time the ground more thoroughly covered.

The distance apart at which transit-lines were run was regulated by the ease and accuracy with which the side levels could be taken. The lines were started 400′ apart at the east edge of section 2 but were at once changed to 300′ apart, which spacing was kept for all of the work, south of the river. On the north of the river, lines were spaced 100′ apart on the east mile and a half of tract and 300′ apart on the west mile and two-thirds, the intervening lines being spaced 150 apart ; the spacing being determined both by the roughness of the ground and by the thickness of the timber growth.

The rate of level-work per party (generally leveler and two rodmen) per day worked was as follows :

Feb. 14-21 ; aver. per party-day = 1.67 miles ; —max. per party-day = 2.0 miles.
“ 22-28 ; “ “ “ = 1.74 “ ; — “ “ “ = 2.2 “
Mch. 1-7 ; “ “ “ = 1.83 “ ; — “ “ “ = 2.3 “
“ 8-14 ; “ “ “ = 2.15 “ ; — “ “ “ = 3.06 “
“ 15-21 ; “ “ “ = 1.80 “ ; — “ “ “ = ——— ——

Only about 7 miles remained after last date, very rough and in section 29; two level-parties were discharged and this finished by the remaining one at the rate of 1.53 miles per day worked. The lowered rate from March 15-21 was due to the work being scattered—"cleaning up."

The total 134.0 miles of levels were run in 72.7 party-days, or at the rate of 1.84 miles per party day.

The increasing speed of work was due to the same general causes as with the transit-work—except that the rodmen were all procured from a distance and were supposed to be good and experienced men, though some of them proved poor.

No care was taken with these levels to have the elevations precise within two or three tenths, many readings being taken by raising rod from the ground and dropping it to ascertain the amount raised. The leveling was started from carefully checked bench marks not over 500′ apart along the bases; run to the river or the lake and checked on the water; then on an adjoining line back to the base, where they closed within a tenth or two, usually much less. The instruments used were ordinary railroad Y levels, the best being a 20″ Gurley several years old; and 12 foot self-reading rods made by ourselves.

Establishment of Bench Marks.—The Datum Plane chosen as the zero of elevations was the U. S. Harbor Datum for Chicago, this being 1 foot above the Chicago City Datum, which is the L. W. of 1847 in Lake Michigan.

The nearest government bench mark was at Hammond, nearly 10 miles away, and this was connected with a primary bench mark chosen at south-west corner of section 4, by three lines of levels run most of the way along the road-bed of the Michigan Central Railroad. The results obtained as the elevation of the primary bench mark were +21.′91, +21.′83, and +21.′84; the last was adopted as being done under the best conditions, the first having been done with an ordinary laborer as rod-man and the leveler reading the clamped rod in passing. About 14 of the turning-points were made permanent and were used in common on the three lines, showing an agreement on intermediate points about the same as on the primary bench mark at Tolleston.

The levels were run once over this distance of about 9½ miles in one full day, the leveler and rodman taking an 8 A. M. train for Hammond and leveling back to Tolleston by about 5 P. M.

As a precaution, the government bench mark at Hammond was verified from one at Hegawisch, about 3½ miles distant, by a single line of unchecked levels; the result was a disagreement of 0.′08 in the two bench marks, which may have been the combined effect of the government and our own errors in leveling. As the only object of the check-line was to check the Hammond bench mark as to probable settlement, the disagreement was disregarded and the government elevation of the Hammond bench mark accepted and used.

From the primary bench mark at the southwest corner of section 4, 56 bench marks were established on trees in the tract, most of them about 500 feet apart along the north and south bases. These bench marks were marked with a number and an elevation, the latter being checked by a double line to within 0.′01 to 0.′03. Elevations were also marked at close of survey on 6 stone monuments and 42 iron monuments, all marking government survey corners.

The instruments used in all bench mark work were the 20″ Gurley Y level already mentioned and a very dilapidated Philadelphia rod, the only target rod on the survey.

Sketching Topography.—Very little of this was done, the methods employed leaving hardly anything to be determined by this means. The edges of sloughs, which had already been located wherever intersected by the regular transit lines, were carefully sketched between ; all of the few buildings were sketched, measured, and connected with the regular transit-lines ; and on the railroads the beginnings and ends of cuts were located, and the depths of cuts and heights of fills noted. Except the railroads, the tract did not contain a single legal road or bridge.

Soundings for Hard Bottom.—The instruments used were a ¾″ gas-pipe with a steel point welded on and a ¾″ auger welded on a ¾″ gas-pipe, the former being most used. In the river, lines of soundings were taken across the stream and flat at about each 1500′; and on sloughs, about 900′ apart—the work in both cases being done on the ice. The river flat was also examined in many places with a spade for clay and some beds of about a foot thick were found, though of no great purity or extent.

Ascertaining the True Meridian.—A true meridian was laid out and monumented north from the south west corner of section 4. The method used was that of observations on Polaris off of meridian, the local time (and consequently the hour angle) being known. Seven observations were made one night, with an extreme variation from mean of 22″; and five another night with a range of 70″ and differing in their mean 1′.08″ from the mean of first night's observations. The mean of the two nights' means was used as the true meridian. Both direct and reverse sights were taken each night and the large range is due mainly to this and to the transit used ; also to the inexperience of the several observers.

This manner of finding the true meridian is worthy of description. It was published, with tables necessary for use, in the U. S. government "Manual of Surveying Instructions" for 1890 and has since, the writer believes, been included in later editions of Hodgman's "Land Surveying." The work of finding the true meridian in this instance will be described, rather than a general case.

The local time was determined by comparing a watch with the 90th meridian time (as telegraphed daily to every railroad office) on the day before and the day after the observation-night, and correcting this 90th meridian time for the difference in longitude as ascertained from a map. The local time of observation being known by the watch and the time of the culmination of Polaris :--from a published table the difference in time is the hour angle of Polaris : from this, using tables or the proper formulae, with the latitude determined from a map, the angle of Polaris with the true meridian at the instant of observation can be computed and the true meridian laid off by this angle.

The advantages of this method are evidently the entire freedom as to time of observation, and the ability to make a great number of separate observations on the same night.

The field-work was done as follows, the problem being to make a rather accurate determination of the true meridian with a transit unsuited to such work :

The point chosen for the transit and for the south monument of the standard meridian was the stone monument already marking the southwest corner of section 4. About 125 feet north of this and on the brow of a hill about 25 feet higher two large stakes were driven very solidly about 5 feet apart east and west, on these stakes a smooth board was nailed, flat with a planed edge to the south. This was done during the day ; also two boxes large enough to hold a lantern provided, one box with a hole in bottom about ⅛" diameter and the hole covered with white paper.

The cross-hairs of transit were lighted by light reflected from a ring of white paper held in front of the object-glass by a wire. Five men and three lanterns were used ; an observer, a time-keeper, and a lantern-holder at the transit, and a man to move box containing lantern, with a helper, on the hill. Merely as an experience, each man was given a chance to act as observer.

The routine of an observation was as follows : the transit, in a direct position, was carefully leveled-up and clamped just ahead of the moving star ; when the vertical cross-hair covered the star, time was called and recorded and the telescope immediately plunged to bring the board on the hill into the field ; the box (set on end) was then moved east or west by signals till the illuminated hole in the bottom was bisected by the vertical cross hair; and then by means of a plumb-bob (the box being held from moving by a weight previously placed on top) the position of the center of hole was carefully marked on the smooth south edge of the board with a pencil.

These observations were repeated on the first night 7 times, using both direct and reverse positions of the telescope, and each pencil mark on edge of board was marked with a number to allow of identification.

On the next day, the angle off the meridian at each observation was computed and the tangent of this angle multiplied by the carefully measured horizontal distance between the south edge of the board and the transit-point; the results were seven distances, which, laid off in the right direction from from their corresponding pencil marks on the edge of the board, gave seven independent determinations of the true meridian, in good shape for comparison and for the choice of a mean or other value.

The adopted position being marked on the edge of board, it was transferred to intersecting threads; the board was then removed and a monument set, and marked from the threads.

The only feature of the foregoing original with the writer (and this has probably been used by others) is the use of the horizontal distance and the tangent of the angle to get a linear instead of an angular determination of the true meridian, and so to confine the use of the transit entirely to plumbing down from the star to the board.

It seems to the writer that a similar method of linear determination might be used for latitude:—transferring the altitudes of Polaris horizontally to a building or a tree by a transit and then computing the angles by means of tape and level measurements; however, the practical difficulties are much greater than in an azimuth-determination and the method is mentioned merely as a possibility in the absence of reliable instruments having vertical arcs.

Boundary Survey.—The object of the boundary survey was to establish a position for every government survey corner (except a few of the meander corners) on the tract mentioned in the original surveys, to use all legal means to make these positions correct and binding, and to occupy every such position with a permanent monument.

As the original surveys were made in 1830-34, the corners being then marked by wooden stakes and referenced to bearing tress, very little evidence of the original marks was in existence in 1891. During the sixty years which had passed no improvements had been made on the tract except the railroads; much of the original timber, including many bearing trees, had been cut down;—the land had always been considered of small value ;—and all incentive for the preservation of the original survey corners seems to have been lacking.

The first step taken was to become personally acquainted with the position of every government survey corner still marked on the tract and with the evidences to which such position was due. This was done with the help of Mr. John Fisher, County Surveyor of Lake County, Indiana, who in the beginning of the survey personally visited each government survey corner with the writer, describing his knowledge of each and pointing out all evidences remaining. Before the close of the survey the writer, in conjunction with Mr. Fisher, prepared a careful statement of all Mr. Fisher's knowledge concerning these survey corners, which statement was entered in "Book No. 1" and was there subscribed and sworn to before a notary public by Mr. Fisher.

The position of county surveyor had at that time been held by him continuously for about 25 years, during which period most of the evidences of the correctness of corners, available during his early years in office, had disappeared. His sworn statement in "Book No. 1" is therefore (except his personal testimony) believed to be the best evidence existing as to the correct location of many of the corners.

It may be mentioned in this connection that the sole condition necessary in any government land survey corner is that it shall occupy the exact spot marked during the original survey ; inaccuracies or mistakes in the original survey have no influence on the subject and do not in the least invalidate the corners then located. From this it follows that the main object in a resurvey is the discovery of the original corner-stake or of some of the original bearing trees for same ; and failing all such evidence, to start from the nearest well-authenticated corners and, with a copy of the notes of the original survey, to re-run this survey so as to locate all missing corners as nearly as possible in their original positions.

A copy was obtained from Mr. Fisher of his certified copy of the field-notes of the original U. S. land surveys. Using this, and his statement of lost evidences, etc., a thorough test and comparison was made of every corner ; such ones as seemed without doubt to be in the original location were accepted and temporarily marked by heavy stakes, while all others were located from these by the use of the original survey-notes as mentioned above.

The re-survey was made much more difficult because of the township line passing east and west through the tract, this, (and the lake on its northern boundary) making every section in the tract a partial section. In many cases it was necessary to go a half mile or a mile off of the tract before a reliable survey corner was found from which to start the resurvey. Many of the locations finally determined on differed widely from those which had been ordinarily accepted as correct —*e. g.*, the position finally determined for the south east corner of section 2 was 27 feet to the west and 9 feet to the south of the previously accepted one, though the latter had been thought worthy of marking with a stone monument.

Sometimes original evidences were discovered after all hope for them had been abandoned. An instance was the south ¼ corner of section 2. A careful search of its supposed neighborhood failed of any result and it was determined to locate it from other survey corners;—a line was run terminating a small swamp hole and a trial stake set, from which (using the compass of the transit) measurements were made to locate the two bearing trees mentioned in the notes of the original survey. At about the spot which should be occupied by the south bearing tree a stump hole was found, burned even with the ground;—while where the north tree should be, a tree was found which had been dead for many years, with a rotten spot on its side which seemed to have been started from a blaze such as is used to mark bearing trees. From these trees were measured lines of the correct distances and bearings, and at their insection, buried 15″ in the wet black muck, was found the point of a stake, 9″ long 3″ square, jelly-like in consistency, but with the ax-marks still visible. The County Surveyor, on examining it afterward, pronounced it to be without doubt the point of the original corner stake.

The writer was greatly aided in this resurvey by two publications:—one a small pamphlet issued by the U. S. Land Office in 1883, called "Restoration of Lost and Obliterated Corners"; and the other a book with the title "Land Surveying", by F. Hodgman, Climax, Mich. The latter contains many legal decisions in reference to surveys and is invaluable on this kind of work.

Provision is made in the Revised Statutes of Indiana, 1881, for making resurveys legal or binding, by an agreement between all interested parties. The essential points to be observed in legal surveys are these:—the survey must be made (at least in name) by the County Surveyor;—all owners of adjoining lands must be notified of date and place of beginning, residents of county by personal service and non-residents by publication for three weeks in the nearest newspaper;—on date of the survey, the monuments can be set only if no objections are made by adjoining owners; a map and descriptions of all monuments set are filed in the county offices;—the cost of the survey is assessed pro rata on all lands bounded by the monuments;—and if no objections are made to the monuments within three years, their correctness cannot thereafter be disputed.

These conditions were all complied with and the survey of tract made a legal survey. Each survey corner established was monumented, usually with a section 4 ft. long of 60 lb. steel railroad rail, set 3′ 6″ in ground, with a bolt through the bottom end. At least two bearing trees were marked and recorded for each monument.

Finally, in "Book No. 1" was entered a complete copy of the field notes of the original government land survey, a signed statement by the writer giving the manner in which each monumented corner was located and the reasons for such location, and a full description of each monument and its bearing trees.

It should be noted that the positions of all monuments were determined and their coordinates obtained by connecting them with the nearest base; and that throughout the boundary survey much use was made of many lines which had been run for the topographical survey.

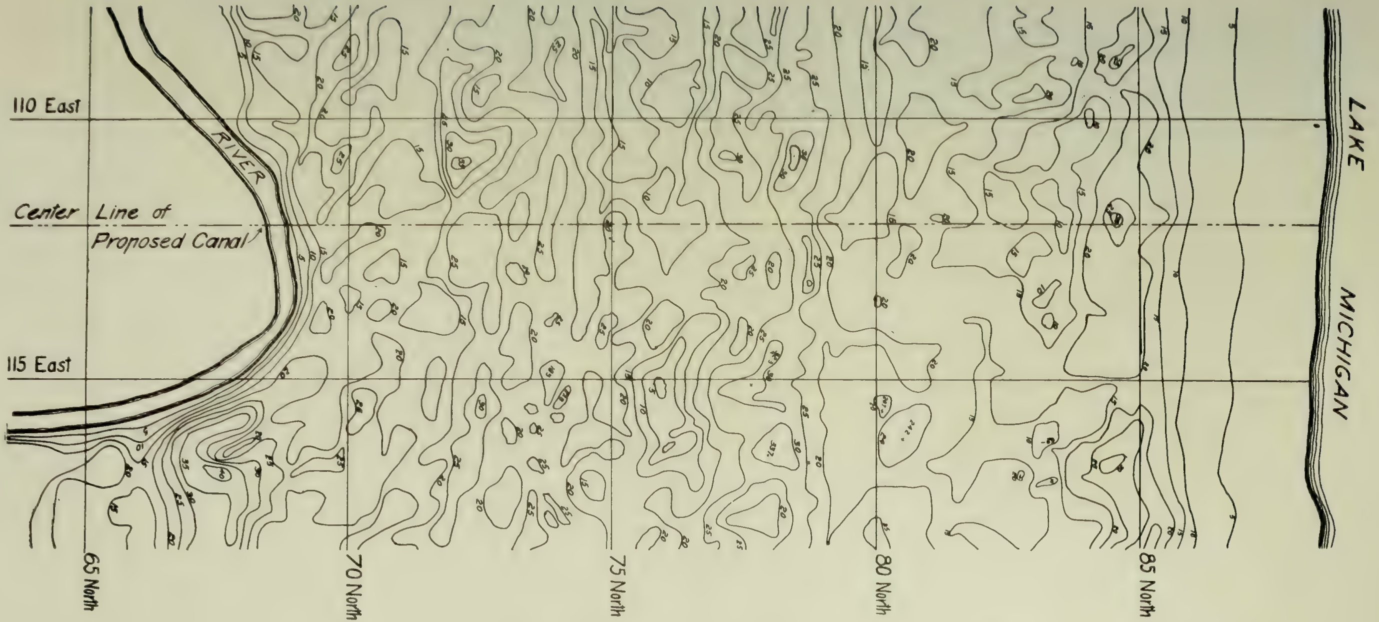
METHODS USED IN MAPPING.

The paper on which the maps were drawn was a thick muslin-backed paper somewhat smoother than "egg-shell paper". Much difficulty was experienced in getting paper wide enough for the purpose, none being found in Chicago and that secured in New York being only 50″ wide. This width, while sufficient to contain the maps, gave very little room for margins, and the appearance of the 100 ft. scale maps was much injured in consequence. It may be of interest to remark that the writer has since found an excellent paper made by A. V. Benoit & Co., of New York, in rolls and 62″ wide; it is called "Samson Paper", is made in two thicknesses, and is especially remarkable for its strength, good surface, and adaptability to erasures.

The drawing-boards (five in all) were made very cheaply and roughly from 1″ boards, by a village carpenter. Their only unusual feature was the trestles, which were made like saw-bucks, but with the two X's joined together by four slats arranged in an hour-glass form. Though built only of 1″ boards and a few nails, they were the steadiest and stiffest the writer has ever seen.

The first step in making the maps was coordinating the paper, or ruling the paper very accurately into squares. The only accurate straight-edge possessed being a 24″ steel one, the main dependence in securing exactly straight lines was on the use of needles and silk threads When the penciled squares had been thoroughly checked and corrected, they were lightly inked with neutral tint, this being used because it was less obtrusive than India ink.

On the eight 100 ft. scale maps, which were drawn first and on which all the field-notes were platted, the squares were made 500 ft. on



SPECIMEN OF CONTOURING

NOTE.
The numbers give height of Contours above Lake in Feet.

a side; while 1000 feet squares were drawn on the general map of 400 ft. scale, which was made by reducing from the original eight.

The great advantages arising from the use of rectangular coordinates in surveys and maps do not seem to be sufficiently appreciated. In no other way can such simple relations be established between all points on the tract, while at the same time computations are greatly simplified, the chance of errors lessened, and maps made much more accurate for future use.

The first surveys platted were the base lines, every section map containing a portion of at least one base This platting was done entirely by the use of computed coordinates, each base station being platted by measuring from the nearest sides of its coordinate square (not more than 250′ at the most) and marked in pencil with its station number.

The two railroads were next platted by computed coordinates in the same way, their surveys being checked by the three base lines which they intersected. The four check lines shown on map were then platted from the bases.

The meander line of river was platted, and after being carefully checked, the edges of river were located by the off-set measurements.

The only topographical surveys then remaining to plat were those connected with the contouring, consisting almost entirely of the north and south parallel transit lines and their accompanying levels. The best realization of the amount of this work can be gained from the fact that there were 134 miles of these transit lines to plat and that the level notes, containing many thousands of elevations, filled about 60 ordinary field books.

The transit lines were platted as follows:—

A pencil line was drawn at right angles to the base at the station from which the transit line was started, and the return line platted by means of the offset at further end and the base station at which the return line terminated. The stations (noted in the field) at which each transit line crossed the railroads, check lines and bases were then marked on the map in pencil and all intermediate stations on the transit lines were fixed by working back and forth from these, distributing inaccuracies at the same time. The pair of lines were platted at the same time as a check on each other, and any large disagreements or errors were noted for looking up in the field. The usual errors (or corrections) on these transit lines have already been given in the description of field work.

When all transit-line stations had been located on the map as closely as practicable to their actual locations in the field, the platting of the level notes was begun. This consisted merely in marking each elevation in pencil at the proper station and offset, keeping a sharp watch for disagreements while doing so. These lines and levels were platted without making use of the coordinate squares, and with bristol board scales.

The only thing still to be done before the development of the contours could be begun was the platting of the edges of sloughs and of the lake shore. These were already located on the map at each transit line; and the sloughs were drawn between these lines from paced sketches.

The sloughs had a very marked effect in determining contours, being long, canal-like bodies of water between well defined ridges, those in the south half of sections 2 and 3 covering about 1⅔ miles east and west and being from 50′ to 700′ in width, while those in section 32 were from 25′ to 150′ wide. Their maximum depth was about 5′; and their surfaces about 18 feet above lake or river in sections 2 and 3 and 7 feet in section 32.

Contours were drawn at 5 foot intervals above the datum plane, being deduced in the usual way by interpolating between the elevations written in pencil on the map. Several small devices can be used to hasten such work, but will not be mentioned here. It was the invariable rule, when water was present, to begin at the lowest contour and work up —and in all cases to work out the lower and upper contours before attempting the intermediate ones.

While the theory of contours is very simple, the writer can testify that deducing the contours for such a piece of ground as this tract is a very difficult work, requiring much skill, care, and patience. The sample of contours given is not selected, but happens to be the only one now at hand. It is from a tracing made at the time of that part of a 100 foot scale map including the proposed canal and (except that the elevations of crests and hollows were omitted) is an exact copy of the original. It also shows the only portion of the tract examined by the writer with completed map in hand; this was done in making studies and borings for a ship canal at this site and the map found to be exact in all respects.

The ground represented by this sample of contours may be regarded as about the easiest and most regular of that covered by the 100 ft. spacing of transit-lines, where this spacing was governed by the roughness of the ground rather than by the thickness of the timber growth. The eastern part of section 35 was many times more broken, while the slough portions of sections 2, 3, and 32 were much more regular.

No further explanations of mapping are needed, except that in reducing the large scale maps by the pantograph, some difficulty was experienced in causing the reduction to fall into its proper position on the general map; it was also found impossible, because of insufficient "reach" in the pantograph, to reduce directly from the middle portions of the large-scale maps, and hasty pencil tracings in strips had to be made for this purpose.

The boundary map, made on the same scale as the general map, was in no way unusual, except that all survey-corners were platted and all distances computed by coordinates.

The "Cost of Surveys" which follows needs no explanations beyond those contained in the statements themselves; the cost is that for the surveys alone, excluding all other work done :

COST OF SURVEYS—BOTH PAY ROLLS AND INCIDENTAL EXPENSES.

I. TOPOGRAPHICAL SURVEY—3,944.17 ACRES.

		COST.	TOTALS.
Superintendence.			
Pay Rolls only		$173 51	
R. R. Fare and Telegrams, 22.68; Stationery and Postage, 4.05		26 73	$200 24
Bases.			
Pay Rolls :—Preparing for Measurement, 12.30 miles, at 16.90	$208 29		
Measuring with Steel Tape, 12.30 miles, at 7.97	98 19		
Computing Corrections, 12.30 miles, at 4.88	59 98		
Measuring Main Base Angles, 8 angles, at 1.48	11 86	378 32	
100′ Steel Band-Chain, 4; Thermometer, .25	4 25		
Spring-balance, .25; tacks, .20; axes, 2	2 45	6 70	385 02
Bench Marks.			
Pay Rolls :—Transferring Elev. from Hammond, 9½ m. run 3 times, 3½ m. run once, .64	20 33		
Establishing B.M.'s on Tract, 56 B.M.'s on trees., .75	42 19	62 52	62 52
Transit-Lines for Locating Contour-Levels.			
Pay Rolls :—Running lines with parties, 134 miles, 5.17,	692 47		
Errors in Stationing—Cost to correct	5 42		
Check-lines on Stationing	48 58		
Traversing R. R.'s—also used as check lines—8.37 miles, 4.85	40 53	787 00	
Transit-rent, 14.10; transit-repairs, 1.40; plumb-bobs, 3.35	18 85		
Sight-poles (4), 8; axes, 6; tacks, .20; nails, .32	14 52		
100′ Surveyor's Chain, 10.35; Lath for station stakes, 11,	21 35		
Red chalk, .10; note books, 2; pencils, .16	2 26		
Express and freight, 7	7 00	63 98	850 98
Levels for Contouring.			
Pay Rolls :—Running lines with parties 134 miles, 3.97,	531 27		
Errors in levels—Cost to correct	40 68	571 95	
Flexible rod, 4; lumber, oil and paint, 2.20	6 20		
Note books, 16; pencils, .50	16 50	22 70	594 65

Topography, etc.			
Pay Rolls:—Meandering river on ice, 6.8 miles line, 7.90	53 65		
Sketching edges of sloughs	71 80		
Sketching buildings	1 72		
Rough measurements of R. R. cuts and fills, 8.37 miles, 1.45	12 14	139 31	139 31
True Meridian by Polaris.			
Pay Rolls—Night observations, 2 nights, 3.28	6 56		
Determining local mag. decl.	34	6 90	6 90
Soundings, etc.			
Pay Rolls:—S'd'g river and flat for hard bottom and depth of water	27 49		
S'd'g slough for hard bottom and depth of water	20 86		
Locating and examining clay beds	7 46		
Taking samples of seepage-water	4 36	60 17	
Gas-pipe rods with auger, 4.60; express, .85	5 45		
Bottles for seepage-water, .40	40	5 85	66 02
Indexing Notes.			
Pay Rolls only	10 00	10 00	10 00
Perpetuating Survey (*see also* "*Boundary Survey*").			
Pay Rolls:—Setting permanent hubs at 82 coordinated points on bases, 82 hubs, at .55	45 51		
Monumenting standard 100 ft.	2 00		
Monumenting true meridian	2 15		
Transferring eleva's to monuments and marking same on 48 monuments, 1.21	58 26		
Computing coordinates of all monuments and of above permanent hubs; and recording coordinates and all bench marks in "Book A".	20 17	128 09	
Monument for true meridian, 60 lb. railroad rail, 4′ long, say	1 50		
Monument for standard 100 ft., 60 lb. railroad rail, 4′ long, say	1 50	3 00	131 09
Mapping Survey.			
Pay Rolls:—(1) 8 sec. maps, scale 1″ = 100′			
Coordinating paper, 500′ squares, 3%	20 00		
Platting field-notes, 41.2%	278 04		
Deducing Contours, 24%	162 36		
Inking contours and lines, 3.6%	24 59		
Lettering and numbering in ink, at Tolleston, 155.37; at Chicago (7 titles), 21.75; 26.2%	177 12		
Final cleaning with bread, etc., 2	12 00	647 11	
(2) General map, scale 1″ = 400′			
Coor. paper, 1000 squares; and platting R. R. and sec. lines, 12%	21 49		
Reducing contours in pencil, mainly by pantograph, 38.5%	69 32		
Inking and numbering contours, 13%	23 75		
Inking R. R.'s sec. lines, etc., 11%	20 06		
Title, etc., by a draftsman in Chicago, 23%	41 25		
Final cleaning with bread, 2.5%	4 20	180 07	
Office rent, 14, and repairs, 2.50; fuel, 4.05; drawing-boards, 27.48	48 03		
Muslin, .76, drawing-paper, 31.76; India ink, etc., 1; pencils, .60	34 12		
Rubber, .40; pantograph, 15; curve pen, 1.58; bread, 1.75	18 73		
R. R. fare and expenses, 12.98; express charges, 5	17 98	118 86	973 04
Total Cost of Topographical Survey			$3,419 77

2. BOUNDARY SURVEY—3,944.17 ACRES.

		COST.	TOTALS.
Superintendence.			
Pay Rolls only	$186 68		
R. R. fare, telegrams, etc.	9 22		$195 90
Field Work.			
Pay Rolls:—Establishing missing corners, etc., 14.7 miles of sec. line, at 39.18	574 90		
Locating corners, mainly from "bases", 14.7 miles of sec. line, 4.11	60 43		
Monumenting corners, 14.7 miles of sec. line, 7.15	104 99		
Legalizing survey—inspecting corners with county surveyor after determining them—14.7 miles of sec. line, 2.98	43 80	784 12	
100′ steel band-chain, 4; transit rent, 10.90; axes, 5.05	19 95		
Tacks, .15; lath for stakes, 1.22; note books, 1.82; pencils, .15	3 34		
Monuments for survey corners—41 R. R. rail, 4′–60, lb. at say 1.50; 6 stone, say 6″ x 6″–4′, at say 3	79 50	102 79	886 91
Mapping, etc.—1 map, scale 1″=400′.			
Pay Roll:—Drawing of map, 41.7%	36 80		
Computing acreage, 14%	12 33		
Statements of monuments, etc., for filing at county seat; copy of same in "Book 1" for proprietors; and tracing, 1″ = 800′, for filing at county seat, 44.3%	39 03	88 16	
Drawing paper, 3; bread, etc., .25	3 25		
Notary's fees, .50; publication of survey notice, 3.60	4 10	7 35	95 51
Total Cost of Boundary Survey			$1,178 32

NOTE.—Under head of "Pay Rolls" in Statement of Cost, are included the following special charges:—

Topographical Survey.		
Team hire—on bases, 6; on transit lines, 10	$16 00	
Boat hire, on transit lines	1 00	
Draftsman in Chicago, 84 hours at .75	63 00	$80 00
Boundary Survey.		
Team hire—on monumenting corners, 2.75; on legal survey, 2	4 75	
Boat hire, on establishing corners	1 00	
County Surveyor, identifying corners (not "legalizing survey") 4 days at 10	40 00	45 75

The item of "Superintendence" in above Statement of Cost is the salary of the writer only while supervising the labor of others, without himself taking any visible part in performing the actual work.

The wages paid during survey were as follows:—Transitmen, $80 per month; levelers, $65 per month; rodmen, $50 per month; head chainmen, $50 per month; all other field men, $1.50 per day; draftsmen, 1 at $85 per month; others at $60 per month.

These wages were total, no board or traveling expenses (except on business errands) being paid.

COST OF SURVEYS—SYNOPSIS.

1. TOPOGRAPHICAL SURVEY—3,944.17 ACRES.

	COST.		
	TOTAL.	PER ACRE.	%
Superintendence	$200 24	$0 051	8 2
Bases	385 02	097	15 7
Bench Marks	62 52	016	2 6
Transit lines for locating Contour levels*	850 98	216	34 8
Levels for Contouring*	594 65	151	24 3
Topography, etc.*	139 31	035	5 7
True Meridian	6 90	002	0 3
Soundings, etc.	66 02	017	2 7
Indexing Notes	10 00	002	0 4
Perpetuating Survey	131 09	033	5 3
Total Surveying	$2,446 73	$0 620	100%
Mapping—8 Section Maps	$674 11	$0 171	69 3
1 General Map	180 07	046	18 5
Incidentals	118 86	030	12 2
Total Mapping	$937 04	$0 247	100%
Total Topographical Survey	$3,419 77	$0 867	

*NOTE.—These lines and levels did not extend over the river-flat (containing 314 acres) and a more accurate cost per "acre covered" would be :

Transit lines = $851 ÷ 3630 acres = $0.235.
Levels = $595 ÷ 3630 acres = $0.164.
Topography = $86 ÷ 3630 acres = $0.024.

2. BOUNDARY SURVEY—3,944.17 ACRES.

	COST.		
	TOTAL.	PER ACRE.	%
Superintendence	$195 90	$0 050	16 6
Field Work	886 91	0 225	75 3
Mapping, etc.	95 51	0 024	8 1
Total Boundary Survey	$1,178 32	$0 299	100%

FINAL SUMMARY OF COST.

		TOTAL.	PER ACRE.	%	%
Topographical Survey.					
Surveys :—Pay Rolls	$2,317 77				
Other Expenses	128 96	$2,446 73	$0 620	71 5	
Maps :—Pay Rolls	854 18				
Other Expenses	118 86	973 04	0 247	28 5	
Total Cost Top. Survey		$3,419 77	$0 867	100%	74 4
Boundary Survey.					
Field Work :—Pay Rolls	$970 80				
Other Expenses	112 01	$1,082 81	$0 275	92 0	
Maps, etc :—Pay Rolls	88 16				
Other Expenses	7 35	95 51	0 024	8 0	
Total Cost Boundary Survey		$1,178 32	$0 299	100%	25 6
Totel Cost of Surveys		$4,598 09	$1 166		100%

In conclusion, the writer does not mean to recommend the methods used in this survey for general adoption, at least not the combination of them here described. He has seen very few tracts of land which could not be surveyed and mapped by much quicker and cheaper methods;—but this paper is offered as a description of what still seems to him the quickest and the cheapest way of obtaining such maps of this special tract as would make it possible to best adapt the tract to the uses proposed.

These very complete maps have been so far of no use, as the Union Stock Yard Company compromised with Armour, Swift & Morris in September, 1891, and these packers agreed to remain at the Chicago Stock Yards. The writer's connection with the tract terminated in September, 1891, his work, in addition to the surveys, including studies for canal, harbor, harbor entrance, docks, grading, etc., and finally a partition of the tract between Armour, Swift, and Morris, and the Union Stock Yard Company.

It may seem that this description of a comparatively small survey is given in too great detail—but the writer can only repeat what he has already said, that he believes it should be of much interest to at least the younger members of the Association.

DESCRIPTION OF THE METHOD OF GAUGING THE DISCHARGE THROUGH THE OUTLET OF HEMLOCK LAKE, N. Y.

JOHN F. SKINNER, C. E , '90.

Hemlock Lake is the source of water supply for the City of Rochester, N. Y. Its outlet is a shallow stream which enlarges into a long, shallow mill pond about half a mile below the foot of the lake.

It was desired to ascertain the discharge of the lake through its outlet at different elevations of the lake surface.

A highway bridge crosses the outlet at the foot of the lake, and a little below a plank channel was constructed 26 feet wide on the bottom with 1½ to 1 side slopes, and 96 feet in length. Its bottom is at an elevation of — 1.00, zero being taken at mean low water or the elevation at which discharge through the outlet, begins. At the upper end of the channel and resting on the bottom a 12 x 12-inch timber crosses it from side to side and retains the earth of the natural stream bed above. Eight (8) feet up stream from the lower end of the channel is placed a weir 0.98 feet high, with a crest 2 inches wide. The elevation of wier crest is — 0.02 ; hence, in general, when water is running through the outlet the weir is submerged. Vertical stop-planks or needles are provided, by means of which the opening at the weir can be contracted to a length of 10 feet. A light bridge spans the lower end of the channel and its trusses form the frame of a housing in which the stop-planks, floats, etc., are kept, when not in use

On the outside of this housing, facing up stream, each foot width of channel is marked and numbered conspicuously, each way from the center ; corresponding numbers are marked on the highway bridge above mentioned so that when a float is liberated an observer on the bridge can line it in laterally to the nearest half foot.

Wires are stretched across the channel at known distances apart and another observer who throws the floats in above the first wire notes their passage of the initial and terminal wires with a stop-watch, calling to the observer on the bridge so that he may note their lateral positions at the instant of passing under each wire. As the velocity is not great, one observer can use two stop-watches and observe two floats at a time by starting one a little in advance of the other.

Four persons are required in making a gauging; the two observers already mentioned, a third who with a hook gauge reads the elevation of the lake surface, the water surface in the channel above the submerged weir and in the slack water below the weir, and who also observes floats along the sides of the channel where the velocity is small, and a fourth person who rescues the floats just before they reach the weir.

The floats are cylindrical wooden rods 1.9 inches in diameter, and loaded so as to float vertically and almost entirely submerged. As they are used repeatedly, they are painted to prevent water soaking. They are of various lengths, and the longest ones which will float without dragging on the bottom are selected for each experiment. It is assumed that these floats give the mean velocities in the vertical planes in which they travel.

The following gauging was made May 24, 1897:

The stop-planks were in position on the sides of the channel, leaving an opening 10 feet wide at the submerged weir.

Wires were stretched squarely across the channel and located as follows: The first, 63 feet up stream from the weir; the second, 15 feet down stream from the first, and the third, 10 feet down stream from the second.

Runs were generally 15 feet from first to second wire, some were 10 feet from second to third wire, and some were 25 feet from first to third wire. All, however, were reduced to the basis of a 25 foot run.

As the water surface was not quiet, maximum and minimum readings were taken to determine the mean elevation. The mean of forty readings gave the elevation of lake surface + 1.362. The mean of forty readings gave the water surface in the channel above the submerged weir + 1.333 or 2.333 feet above the bottom of the channel, and the same number of readings gave the elevation of the water surface below the weir + 1.171 or 2.171 feet above the bottom of the channel.

As there was considerable silt in the channel, soundings were taken as shown in Table I.

Width of water surface in channel above weir $= 26 + (3 \times 2.333) = 33.00'$ feet.

TABLE I.

Distance in Feet Right or Left from center of Channel.		DEPTH OF WATER IN FEET.			
		First Wire.	Second Wire.	Third Wire.	Mean Section.
16.50	left.	0.00	0.00	0.00	0.00
14.85	"		1.10		1.10
14	"	0.99	1.20	1.35	1.18
13	"	1.03	1.26	1.36	1.23
12	"	1.09	1.27	1.37	1.25
11	"	1.13	1.30	1.43	1.29
10	"	1.13	1.27	1.42	1.27
9	"	1.20	1.38	1.46	1.35
8	"	1.31	1.41	1.67	1.45
7	"	1.48	1.47	1.70	1.53
6	"	1.56	1.51	1.73	1.58
5	"	1.67	1.62	1.77	1.67
4	"	1.74	1.66	1.83	1.72
3	"	1.78	1.68	1.78	1.73
2	"	1.90	1.76	1.88	1.82
1	"	1.96	1.87	1.85	1.89
0	center.	1.96	1.81	1.76	1.83
1	right.	2.00	1.86	1.87	1.90
2	"	2.07	1.92	1.89	1.95
3	"	2.07	1.95	1.92	1.97
4	"	2.12	2.00	1.90	2.00
5	"	2.13	2.00	1.98	2.03
6	"	2.14	2.07	2.02	2.07
7	"	2.15	2.05	2.07	2.08
8	"	2.12	2.04	2.03	2.06
9	"	2.12	2.05	1.99	2.05
10	"	2.06	2.00	2.00	2.01
11	"	2.04	2.00	1.97	2.00
12	"	1.96	1.96	1.98	1.99
13	"	1.85	1.92	1.90	1.90
13.85	"				1.76
16.50	"	0.00	0.00	0.00	0.00

Two hundred and forty-five (245) floats were observed, and their times are plotted in Fig. 1 as ordinates. A mean time curve was drawn and ordinates of this curve at each foot width of mean cross-section of channel are given in Column 2, of Table II. Column 4 gives areas of vertical strips 1 foot wide into which the entire cross-section is divided. Column 3 is the mean velocity in each strip, corresponding to the ordinate of the time curve through the center of gravity of each strip of area. Column 5 contains the discharge through each of said strips.

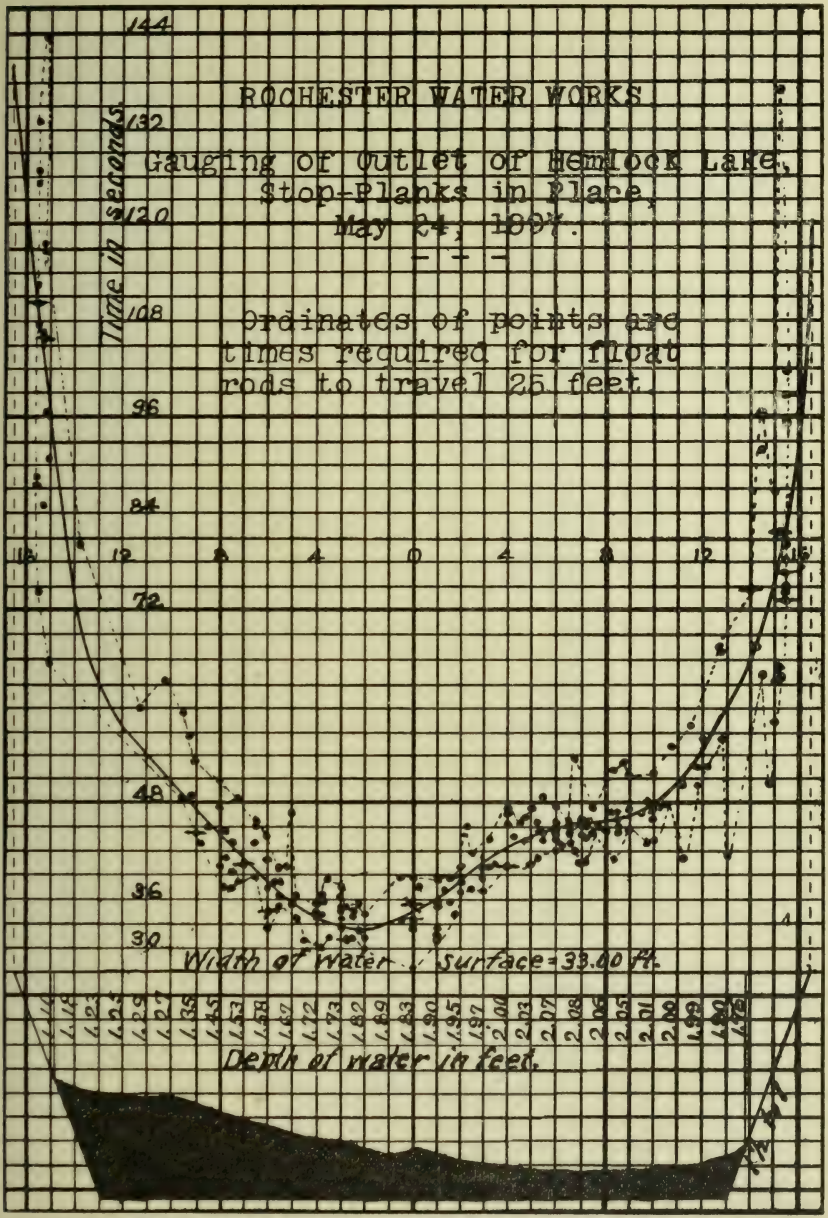

ROCHESTER WATER WORKS.
Gauging of Outlet of Hemlock Lake,
Stop-Planks in Place,
May 24, 1897.
Ordinates of points are times required for float rods to travel 25 feet.
Time in seconds.
144
132
120
108
96
84
72
48
36
30
16
12
8
4
0
4
8
12
16
Width of Water Surface = 33.00 ft.
1.14
1.18
1.23
1.25
1.29
1.27
1.35
1.45
1.53
1.58
1.67
1.72
1.73
1.82
1.89
1.83
1.90
1.95
1.97
2.00
2.03
2.07
2.08
2.06
2.05
2.01
2.00
1.99
1.90
1.78
Depth of water in feet.

FIG. 1.
]

TABLE II.

— 1 — Distance in Feet Right or Left from Center of Channel.	— 2 — Ordinate of Time Curve t. (in seconds for 25 ft. run.)	— 3 — Mean Velocity in Vertical $V = \frac{25}{t}$ (ft. per sec.)	— 4 — Area of Strip 1 foot wide. A. (sq. ft.)	— 5 — Discharge Q = Av. (cu. ft. per sec.)
16.50 left.	140	——	0.00	0.0000
15.83 "	116	0.22	0.33	0.0726
14.96 "	95	0.26	0.97	0.2522
14 "	74	0.34	1.18	0.4012
13 "	63½	0.39	1.23	0.4797
12 "	58¼	0.43	1.25	0.5375
11 "	54	0.46	1.29	0.5934
10 "	50¼	0.50	1.27	0.6350
9 "	47	0.53	1.35	0.7155
8 "	43¾	0.57	1.45	0.8265
7 "	40¾	0.61	1.53	0.9333
6 "	38¼	0.65	1.58	1.0270
5 "	35½	0.70	1.67	1.1690
4 "	33¾	0.74	1.72	1.2728
3 "	32¾	0.76	1.73	1.3148
2 "	32¼	0.78	1.82	1.4196
1 "	33	0.76	1.89	1.4364
0 center	34¼	0.73	1.83	1.3359
1 right	36	0.69	1.90	1.3110
2 "	38¼	0.65	1.95	1.2675
3 "	40½	0.62	1.97	1.2214
4 "	42½	0.59	2.00	1.1800
5 "	43¾	0.57	2.03	1.1571
6 "	44¾	0.56	2.07	1.1592
7 "	45¼	0.55	2.08	1.1440
8 "	45½	0.55	2.06	1.1330
9 "	46	0.54	2.05	1.1070
10 "	47½	0.53	2.01	1.0653
11 "	50¼	0.50	2.00	1.0000
12 "	54¾	0.46	1.96	0.9016
13 "	59¾	0.42	1.90	0.7980
13.97 "	65½	0.38	1.63	0.6194
14.94 "	73¼	0.34	1.00	0.3400
15.83 "	86	0.29	0.33	0.0957
16.50 "	120	——	0.00	0.0000
Total for entire cross-section ----------------			53.03	29.9226

The mean velocity for entire cross-section $= \frac{29.9226}{53.03} = 0.564$ feet per second. By inspection of Table II, it will be seen that the maximum of the mean velocities in the various strips is 0.78 feet per second; hence the ratio of mean velocity in the entire cross-section to maximum mean velocity therein $= \frac{0.564}{0.78} = 0.7233$. The wet perimeter of the section $= 33.86$ feet, and the hydraulic mean radius $= \frac{53.03}{33.86} = 1.57$ ft.

Figure 5 illustrates the conditions at the submerged weir.

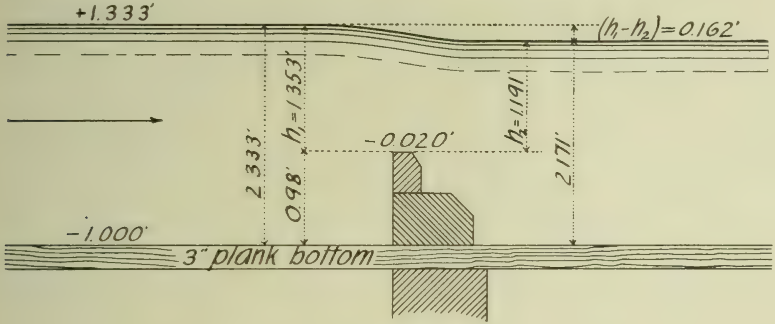


FIG. 5.

Francis' formula for submerged weirs, (neglecting end contractions) is:—$Q = 3.33 \times l \sqrt{h_1 - h_2}\ (h_1 + 0.38 : h_2)$; and with $l = 10$; $h_1 = 1.353$; $h_2 = 1.191$; $(h_1 - h_2) = 0.162$ we have $Q = 3.33 \times 10 \times \sqrt{0.162}$ $(1.353 + 0.454) = 24.22$ cubic feet per second.

Bornemann's formula for submerged weirs is:—$Q = \mu\, l \times 5.35 \sqrt{h_1 - h_2}\,(h_1 + 0.5h_2)$ in which $\mu = \left[0.702 - 0.2226 \sqrt{\frac{h_1 - h_2}{l}} + 0.1845 \left(\frac{h_2}{h_1}\right)^2\right]$

Substituting the above values for l, h_1 and h_2 in the latter expression we have $\mu = \left[0.702 - 0.2226 \sqrt{\frac{0.162}{10}} + 0.1845 \times \left(\frac{1.191}{1.353}\right)^2\right] = 0.817$, whence $Q = 0.817 \times 10 \times 5.35 \sqrt{0.162} \left(1.353 + \frac{1.191}{2}\right) = 34.28$ cubic feet per second.

It will be seen that the float measurement gave a discharge of 29.92 cubic feet per second, while the weir formulas gave in one case, 24.22 and in the other, 34.28. Evidently when accepted formulas differ so widely, the conclusion to be drawn is that they are not applicable to the case in hand, where $(h_1 - h_2)$ is small relative to the length of the weir; also where the crest and ends of the notch are not cut from a theoretically thin plate, thus making the contractions imperfect; also where the most perfect appliances are not available for accurate observation of the elevation of water surface, and where the surface is con-

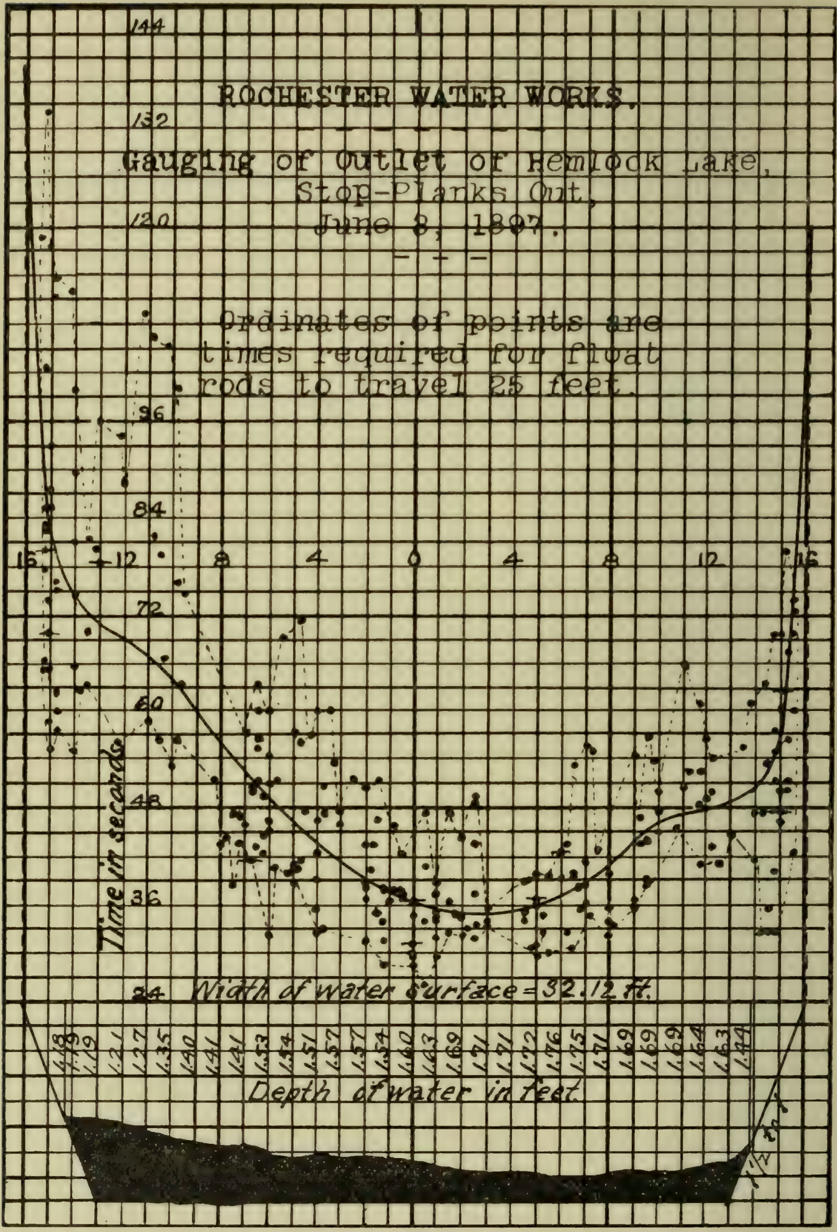

ROCHESTER WATER WORKS.
Gauging of Outlet of Hemlock Lake,
Stop-Planks Out,
June 8, 1897.
Ordinates of points are
times required for float
rods to travel 25 feet.
144
132
120
96
84
72
60
48
36
24
16
12
8
4
0
4
8
12
16
Time in seconds
Width of water surface = 32.12 ft.
1.18
1.19
1.19
1.21
1.27
1.35
1.40
1.41
1.41
1.53
1.54
1.51
1.57
1.57
1.54
1.60
1.63
1.69
1.71
1.71
1.72
1.76
1.75
1.71
1.69
1.69
1.69
1.64
1.63
1.44
Depth of water in feet.
1½ to 1

FIG. 2.

stantly oscillating, due to waves in the lake and gusts of winds on its surface.

Another gauging was made June 3, 1897; 280 floats were observed and their times were plotted as shown in Figure 2. In this case the stop-planks were removed, giving an unobstructed channel. The following results were obtained by a method identical with the case described above: —

Mean elevation of lake surface for the day + 1.080; mean elevation of water surface in the channel above the submerged weir + 1.039; area of mean cross-section of channel, 46.24 square feet; discharge, 25.75 cubic feet per second; mean velocity in entire cross-section, 0.56 feet per second; maximum of the mean velocities in the various strips, 0.71 feet per second; ratio of mean velocity in entire cross section to maximum mean velocity therein, 0.789; wet perimeter, 32.91 feet; hydraulic mean radius, 1.41 feet.

On June 8, 1897, two gaugings were made, one in the forenoon with the stop-planks removed, and one in the afternoon with the stop-planks in place, thereby reducing the channel to a width of 10 feet at the weir. One hundred and four floats were observed in the forenoon and 135 in the afternoon. Their times were plotted and are shown respectively in Figure 3 and Figure 4.

Mean elevation of lake surface for the day + 0.932; mean elevation of water surface in the channel above the submerged weir in the forenoon + 0.900; mean elevation of water surface in the channel below the submerged weir in the forenoon + 0.893; area of mean cross-section of channel, 44.65 square feet; discharge, 21.19 cubic feet per second; mean velocity in entire cross-section, 0.47 feet per second; maximum of the mean velocities in the various strips, 0.63 feet per second; ratio of mean velocity in entire cross-section to maximum mean velocity therein, 0.745; wet perimeter, 32.49 feet; hydraulic mean radius, 1.37 feet.

In the afternoon:—Mean elevation of water surface in the channel above the submerged weir, +0.904; mean elevation of water surface in the channel below the submerged weir, +0.796; area of mean cross-section of channel, 43.17 square feet: discharge, 18.32 cubic feet per second; mean velocity in entire cross-section, 0.42 feet per second; maximum of the mean velocities in the various strips, 0.55 feet per second; ratio of mean velocity in entire cross-section to maximum mean velocity therein, 0.80; wet perimeter, 32.42 feet; hydraulic mean radius, 1.33 feet.

By inspection of the diagrams it will be seen that the time curve is irregular and influenced by a number of conditions.

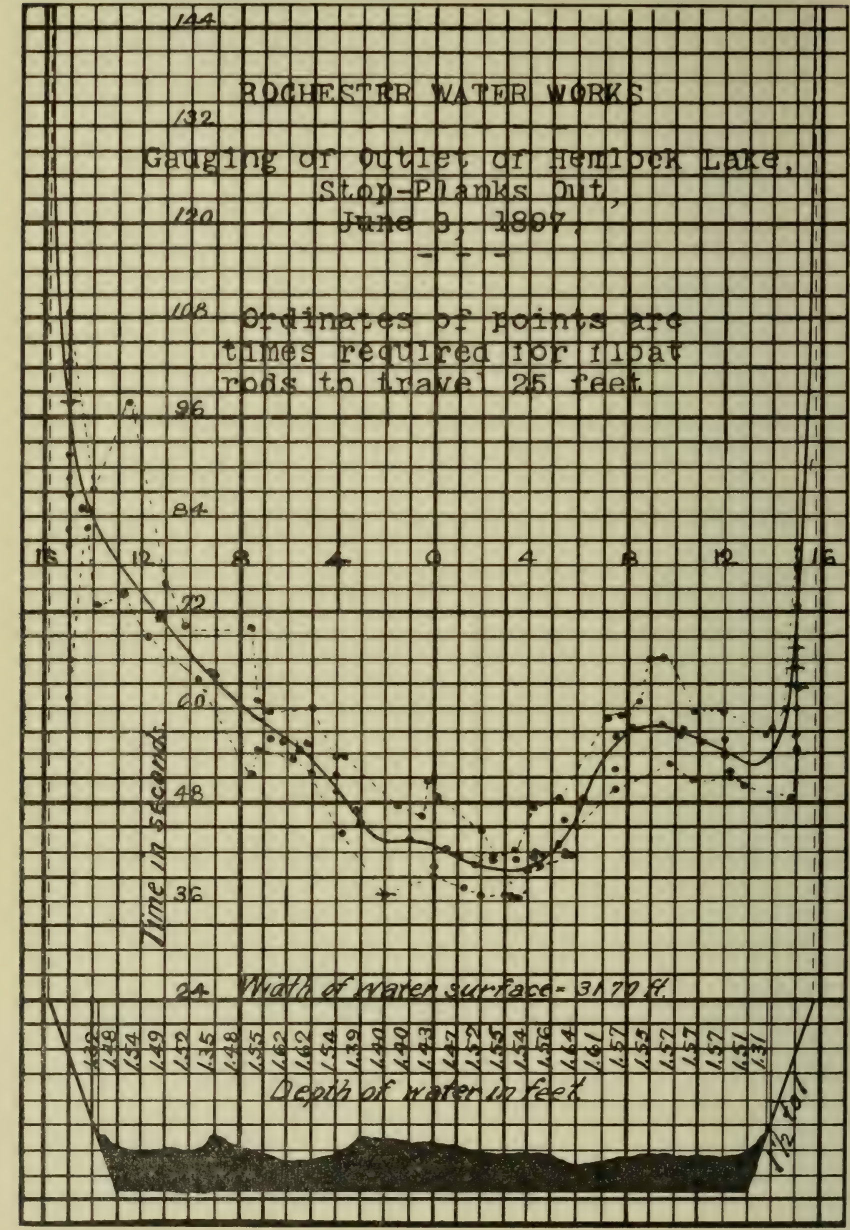

ROCHESTER WATER WORKS
Gauging of Outlet of Hemlock Lake,
Stop-Planks Out,
June 3, 1897.
- - -
Ordinates of points are
times required for float
rods to travel 25 feet
144
132
120
108
96
84
72
60
48
36
24
16
12
8
4
0
4
8
12
16
Time in Seconds
Width of water surface = 31.70 ft.
1.32
1.48
1.54
1.49
1.52
1.35
1.48
1.55
1.62
1.62
1.54
1.39
1.40
1.40
1.43
1.47
1.52
1.55
1.54
1.56
1.64
1.61
1.57
1.55
1.57
1.57
1.57
1.51
1.31
Depth of water in feet
1/2 to 1

FIG. 3.

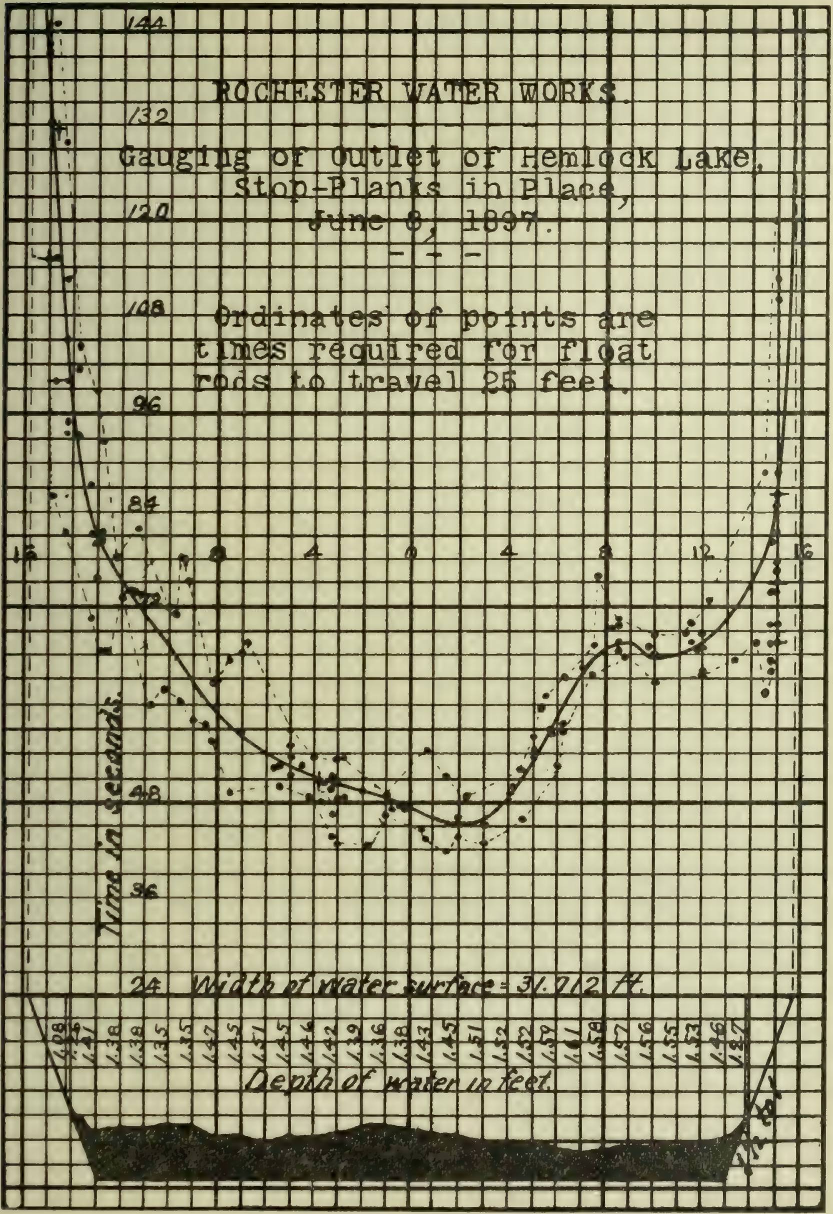

ROCHESTER WATER WORKS.
Gauging of Outlet of Hemlock Lake,
Stop-Planks in Place,
June 8, 1897.
Ordinates of points are
times required for float
rods to travel 25 feet.
144
132
120
108
96
84
48
36
24
16
8
4
0
4
8
12
16
Time in seconds.
Width of water surface = 31.712 ft.
1.08
1.21
1.38
1.38
1.35
1.35
1.47
1.45
1.51
1.45
1.46
1.42
1.39
1.36
1.38
1.43
1.45
1.51
1.52
1.52
1.59
1.61
1.58
1.57
1.56
1.55
1.53
1.46
1.27
Depth of water in feet.

FIG. 4.

When the stop-planks are in, the flat central portion of the curve is is narrower, showing that the maximum velocity is closely confined to the center of the channel.

In the diagrams an elevation in the time curve occurs near the right side. This is caused by a diminished current, due to a sand bar formed near the head of the channel and a little distant from the right bank. The disparity between results which might be obtained from single observations can be readily seen by noting the different times of floats in the same vertical, often differing by one hundred per cent. Again, as the time curve is irregular, it becomes necessary to define it by observing floats in a great many verticals of the cross-section. Hence it is needful to observe as large a number of floats as possible in as many verticals as possible, and it is well to have the gauging extend over a considerable period of time, for the conditions are constantly varying. A little wind up or down the channel will induce a surface current which will retard or accelerate the velocity of the float, and with a gust of wind blowing across the channel, a float may be carried from the side to a position near the center, where the velocity is greater, or *vice versa*.

When the water is low, the greatest eccentricity exists in the behavior of the floats. On one side the current may be running up stream, in which case the negative as well as the positive quantity must be computed, the discharge being their algebraic sum.

In justification of this method of gauging shallow streams it may be said that the floats were here more convenient than a current meter, where so many different positions and observations are required, and where the water carries at times much matter in suspension by which a current meter would soon become clogged and useless.

The submerged weir, under small heads, has been shown to be unreliable ; and, finally, in the method described no empirical formula whatever is used in the calculation. Experiments were made last summer and they are being repeated this year, but not enough results have as yet been obtained to warrant the formulating of any law of the relations existing between the discharge and the hydraulic mean radius, mean velocity or center velocity.

BRICK PIERS

RESULTS OF TESTS MADE IN THE LABORATORY OF THE COLLEGE OF CIVIL ENGINEERING, CORNELL UNIVERSITY, 1897–8.*

The following tests were made with the object of determining the resistance to crushing, offered by piers of common building brick laid up in the ordinary manner of brick walls.

The brick used were manufactured at Newfield, N. Y., and were of medium hardness and of light buff color. Two lots were used, varying somewhat in hardness and are designated respectively as No. 1 and No. 2. The mortar used in construction was made of Portland cement, Giant brand, and was mixed with clean sharp sand in the proportion of 1 : 2.

For testing the strength of the mortar, two sets of briquettes were made. The first set (11 briquettes) was made of mortar quite wet, and the briquettes were allowed to harden in air for two days and were then kept in water for a period of 98 days. The average of the 11 tests gave an ultimate tensile strength of 206.5 lbs. per square inch.

The second set (18 briquettes), made of stiffer mortar, was allowed to harden one day in air, and 94 days in water.

An average of the 18 tests gave a tensile strength of 226.8 lbs. per square inch. An average of these two results was taken as representing the strength of the mortar, the figures being 219.1 lbs. per square inch.

Tests were also made of the compressive strength of single brick lying flat and properly bedded with plaster of paris to insure an even bearing.

An average of four tests on Brick No. 1 indicated an ultimate crushing strength of 3,270 lbs. per square inch, and the average of the same number of brick, No. 2, showed a resistance of 3,750 lbs. per square inch.

The piers were built by a careful bricklayer and were set aside in the laboratory to harden, in a temperature ranging, probably, between 60° F. and 70° F. They were built up on a smooth iron plate, and when placed in the machine for testing, were carefully trued up with building paper, and the top of the pier bedded in plaster of paris and allowed time to harden

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*These tests were begun and part of the material furnished by Messrs. H. N. Ogden and C. W. Comstock, instructors in this College. The work was completed by the class in testing materials as part of their regular laboratory work under the direction of Mr. E. J. McCaustland.

The machine used was a Riehlé of 400,000 lbs capacity and the load was applied very slowly but continuously.

The piers averaged about 13″ x 13″ in section, and the bond showed a whole brick and a half brick on each face, the center having been filled with a half brick or with spawls and mortar. The general method of failure was by splitting in long irregular vertical cracks, as shown in the accompanying view, indicating a lack of tensile strength in the material. In only one or two cases was there any evidence of actual crushing of the brick, and in those cases the crushing occurred at the top of the pier, and was probably caused by a lack of uniform bearing in the machine.

Pier No 1. The failure was most marked at the center of the pier and the cracks followed the vertical joints.

Pier No 2. The bottom ten courses remained intact after the upper part had completely failed.

Pier No. 3. Failed suddenly, with loud report, at 172000 lbs.

Pier No. 4. Showed careless workmanship in construction.

Pier No. 6. Cracked from bottom up. Two top courses intact.

Pier No. 10. Spalling and crushing of brick after maximum load was reached.

Pier No. 12. Vertical cracks full length. Mortar squeezed out.

Pier No. 14. Cement crushed out, without spalling brick, at 108,000 lbs.

Pier No. 15. Failure confined entirely to upper half of pier.

Pier No. 17. Cracks did not follow joints to any extent.

Pier Nos. 1 to 6 were made of brick No. 1, crushing strength 3270 lbs. per square inch.

Piers Nos. 7 to 18 were made from No. 2 Brick, crushing strength 3,570 lbs. per square inch.

The compression for Piers Nos. 14, 16, 17 and 18 was measured between bearing blocks.

No visible cracks appeared in Nos. 13, 14, 15 before the ultimate load was reached.

A study of the results would indicate that greater strength could not be obtained by the use of better mortar or of brick having a greater crushing strength merely, nor by shortening the pier within limits as the effect of length is not apparent.

A set of experiments is under way to determine the effect of the tensile strength of the brick upon the strength of the pier, and also the effect of increasing the lateral strength in tension at intermediate joints by the use of metal.

The following table, somewhat modified from a similar one presented in a laboratory report by Mr. W. J. Graves, gives in a condensed form the principal facts with reference to the construction, the condition of the tests and the method of failure of the entire set :

COMPARATIVE RESULTS OF TESTS OF BRICK PIERS.

College of Civil Engineering, Cornell University, 1898.

	Dimensions Cross-section 13″×13″.				Conditions of Test.		Load in pounds.				Compression.		
									Ultimate.				
	Ht. Feet.	No. Courses.	Av. Bed Joint	Age in Mo's.	Prelim. Load.	Time of Application.	First Sound.	Cracks Visible	Gross.	Lbs. per sq. in.	Total in Feet.	Per Foot.	Ratio to Strength of Brick.
1	7½	34	0.22″	14⅔	5800 lbs.	30 min.	80000	145000	145000	859.0	0.01 in 5	0.0020	0.26
2	7½	"	"	"	8000	20 "	72000	102000	149500	885.0			0.27
3	7½	"	"	"	6800	15 "	160000	172000	172000	1017.0	0.011 in 6	0.0018	0.31
4	6½	29	0.23″	14½	10000	15 "	65000	113000	133400	790.0			0.24
5	6½	"	"	"	5400	15 "	58000	90000	108000	639.5			0.20
6	6½	"	"	14⅔	8000	15 "	72000	122000	124000	734.0	0.02 in 5.7	0.0035	0.22
7	5½	25	0.22″	5	700		91700	156760	156760	927.5			0.24
8	5½	"	"	4½			136500	162420	179500	1062.1			0.28
9	5½	"	"	4	550		15000	79500	146170	864.9			0.23
10	4½	20	0.24″	3½			98400	106850	151590	897.0			0.24
11	4½	"	"	"	2500		80000	163000	184700	1092.9			0.29
12	4½	"	"	"	2500		90000	120000	151200	894.6			0.24
13	3½	15	0.28″	13⅓	4000	10 min.	70000		130000	773.8			0.21
14	3½	"	"	14⅓	10000	24 hrs.	75000		108000	635.3	0.026 in 3.5	0.0074	0.17
15	3½	"	"	14¼			90000		160000	898.8			0.24
16	2½	11	0.26″	14				135000	165000	959.3	0.006 in 2.5	0.0024	0.26
17	2½	"	"	14			112000	126000	147600	873.3	0.006 in 2.5	0.0024	0.23
18	2½	"	"	14				54000	142000	832.2	0.006 in 2.5	0.0024	0.22

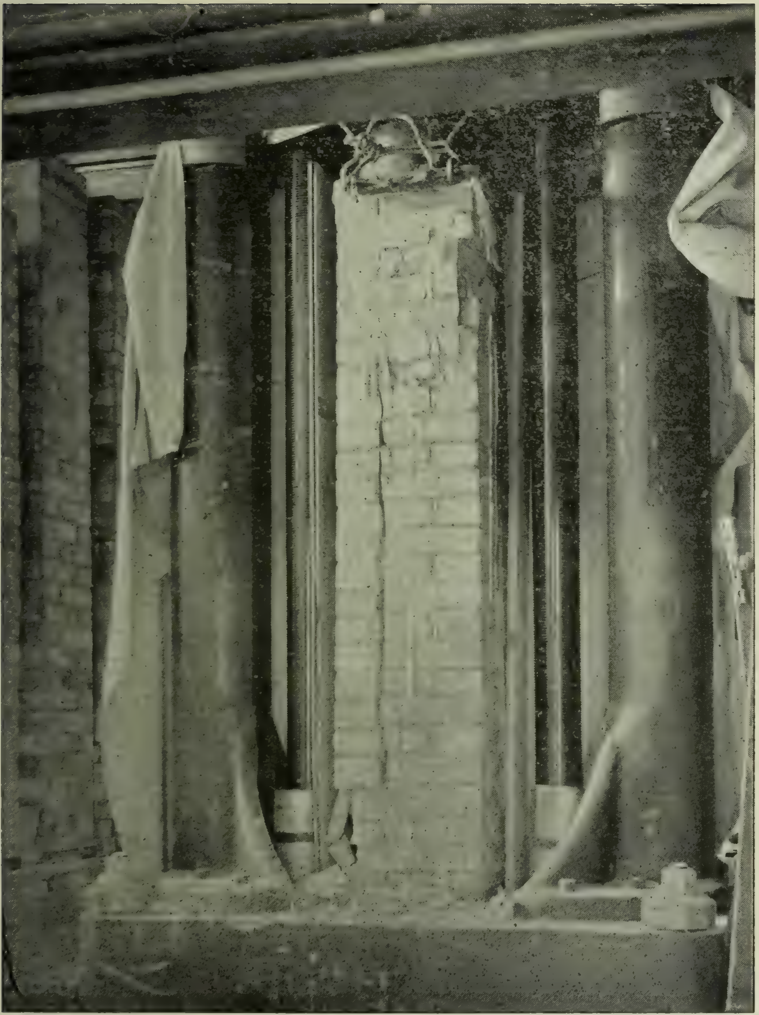

Photo Exhibiting Method of Failure of Brick Pier.

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ELLIS, THEO. G. "Causes of formation of bars at mouths of rivers, as shown in an examination of the Connecticut river." Trans. Am. Soc. C. E., May 21, 1873, pp. 313, II.

ROBERTS, ELLIS, WHITCOMB, SMITH. "River mouths." Discussion. Trans. Am. Soc. C. E., Vol. 4, 1875, pp. 290–297.

BARNARD, J. G. "The delta of the Mississippi,—considered in relation to an open river mouth." Trans. Am. Soc. C. E., March 15, 1875, p. 104.

KNAPP, HENRY F. "Harbor bars; their formation and removal." Trans. Soc. Engs., 1878, pp. 139–166.
Action of waves in transporting material.

WHEELER, W. H. "Theories as to the cause of bars." Proc. Inst. C. E., Vol. 100, pp. 116–216, etc.
Valuable.

CONROY, G. E. "Étude pratique sur les marées fluviales et notament sur le mascaret." Text 8°, 2 Vols., Atlas. Paris, 1881. Ganthier-Villars.
Tidal-waves in rivers; curves showing their form. The regime of different French rivers.

BROWNE, W. R. "Relative value of tidal and upland waters in maintaining rivers, estuaries and harbors." Proc. Inst. C. E., April 26, 1881. Vol. 66, pp. 1–85.

BOUQUET DE LA GRYE, A. "Étude sur le régime de la Loire maritime." Pamphlet in 8°. Paris, 1882. Dunod.

Vernon-Harcourt, L. F. "Rivers and canals." Text 6 × 8, 350 pp., 2d Vol. Plates. Oxford, 1882. Clarendon Press.

A standard English treatise, discussing tidal estuaries.

—— "The principles of training rivers through tidal estuaries." Pamphlet in 8^{o}. London, 1889.

Experiments made with a model of the Seine estuary, to determine the action of river walls.

—— "Les conditions physiques agissant sur les rivières a marée et les principes applicable à leur amélioration." Pamphlet in 8^{o}, 1890. English edition also.

Lyon, Max. "Étude sur l'amélioration de la barre de Rio Grange de Sul (Brésil)." Text 5 × 8, 205 pp. Paris, 1892. A. Maulde et C^{ie}.

Notes on fascine work.

Vernon-Harcourt, L. F. "Memoire sur l'amélioration de la partié maritime des fleuves, y compris leurs embouchures." Pamphlet in 8^{o}. Paris, 1892. Lahure.

Characteristics of tidal and non-tidal rivers. Discussion of working models.

Partiot, H. L. "Étude sur les rivières a marée et sur les estuaries." 1 Vol., 125 pp. Paris, 1892. Baudry & C^{ie}.

The theoretical transmission of tides. A description of the Seine, Garonne, Gironde, Loire, etc.

Corthell, C. E. "Étude sur l'amélioration de l'embouchure des fleuves, principalement en Amérique." Pamphlet, 8^{o}. Paris, 1892. Lahure.

The jetties for the Mississippi, Brazos, Tampico, and Danube rivers.

Franzius. "Communication sur l'amélioration des fleuves dans leur partié maritime." Pamphlet, 8^{o}. Paris, 1892. Lahure.

The method used in training the Weser.

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"Tidal estuaries." Proc. Inst. Civil Eng. Various papers.

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Ellet, Charles. "The Mississippi and Ohio river inundations." Text 8^{o}. Philadelphia, 1853.

Belgrand. "Études des lois qui regissent les crues des cours d'eau." Pamphlet, 8^{o}. Versailles, 1853. Beaujeune.

Discussion of the permeability of the soil and its relation to floods in the basin of the Seine.

Vallès, M. F. "Études sur les inondations." Text. Paris, 1857. Victor Dalmont.

An extended discussion of rainfall phenomena, floods, their prevention, and the utilization of their force.

Monestier-Savignat, A. "Études sur les inondations." Text 5 × 8, 550 pp. Plates. Paris, 1858. Dalmont.

A theoretical discussion of the laws of floods.

Vallès, M. "Inondations." Annales des Ponts et Chaussées. 1860, i, 137.
The solid discharge of the Durance.

Conroy, G. E. "Renseignements sur les crues de la Loire et de ses grandes affluents. Reglès pratique pour determiner à l'advance, sur différent points du littoral, la hauteur et l'epoque du maximum des crués de la Loire." Text 4°. Paris, 1861. Dunod.
A collection of tabulated data giving relative flood heights along the river Loire.

Kleitz, C. "Note sur la theorie du mouvement non permanent des liquides et sur son application à la propagation des crués des rivières." Pamphlet, 8°. Paris, 1877. Dunod.
Analytical treatment of non-permanent motion as applied to floods in the Rhone.

Deutsch, J. "Die überschwemmungen und ihre ursachen." Pamphlet, 100 pp. Vienna, 1877. Lehmann & Wentzel.
Floods of the Danube and their regulation.

Descola, E. "La question des montagnes et les inondations." One pamphlet in 8°. Foix, 1877.
A popular discussion of legislation affecting forest preservation and the rights of mountaineers.

Minister des travaux publics. "Bassin de la Garonne. Reglements et instructions concernant l'annonce des crués et l'étude du régime des rivières." Text 4°. Paris, 1879. Impr. nationale.
A volume of regulations and forms of announcements used in predicting the floods of the Garonne.

Pasqueau, A. "Note sur les glaces de la Saone en 1879–1880." Pamphlet, 8°. Paris, 1880. Dunod.
Use of dynamite in removing an ice jam.

Gustav, Wex. Translated by G. Wietzel. "Decrease of water in rivers." Two Pamphlets, 50 pp. each. Washington, 1881. Government printing office.
The decrease of water in rivers contemporaneously with an increase in the height of floods in cultivated countries.

Lanfranconi, E. "Rettung Ungarns vor ueberschwemmungen." Text 15 × 20, 80 pp., two large maps. Budapest, 1882.
The floods of the Danube in Hungary.

Johnson, J. B. "Floods on the Lower Mississippi as illustrated in the flood of 1882." Trans. Assoc. of Eng. Soc., Vol. 2, p. 115.

Edlen, Karl S. "Ueberschwemmungen." Text 5 × 10, 150 pp. Vienna and Leipzig. A. Hartleben, 1883.
A record and general description of floods.

Ockerson, J. A. "Earlier floods in the lower Mississippi." Trans. Assoc. of Eng. Soc., 1885, Vol. 4, pp. 87–95.

"Règlements et instructions concernant l'annonce des crués et l'étude du régime des rivières." Text 4°. Paris, 1886, Impr. Nationale.
Regulations and forms used in foretelling the Seine floods.

ALLARD, E. "Note sur la prévision des crués." Pamphlet, 8°. Paris, 1889. Dunod.

A graphic method of predicting floods together with table comparing predictions and measurements on French rivers.

STARLING, WM. "Discharge of the Mississippi." Trans. Am. Soc. C. E., 1895, Vol. 34, pp. 348–492, with extended discussion.

A comprehensive and admirable discussion of the discharge of the Mississippi at all stages.

GENERAL HYDRAULIC CONSTRUCTION.

SILBERSCHLAG, JOHANN ESAIAS. "Hydrotechnik oder der wasserbaues." Two Vols., Text 5 × 8, 400 pp. each. Copper cuts. Leipzig, 1773. Caspar Fritsch.

Mainly of historic interest.

WIEBEKING, C. F. "Wasserbau." 3 Vols., Text 9 × 12, 750 pp. each, and one Vol. 250 pp., containing additions. Munich, 1811–1814. Joseph Zäng.

The most comprehensive treatment of the general subject published in its time.

HAGEN, G. "Beschreibung neuer wasserbauwerke." Text 5 × 8, 300 pp. Königsberg, 1826. Gebruder Bomträger.

Description of the larger hydraulic constructions in Germany, France, The Netherlands, and Switzerland.

GUDME, A. C. "Wasserbaukunst." Text 6 × 9, 3 Vols. Berlin, 1827–1829. August Rücker.

An extended work covering all branches of hydraulics with special reference to rivers, harbors, and dykes.

DIETLEIN, J. F. W. "Grungzüge der vorlesungen über Schleusen." Text 8 × 12, pp. 160 to 390. Berlin, 1832. G. Reimer.

PECHMANN, HEINRICH. "Praktische anleitung zum flussbaue." Text 5 × 7, 2 Vols., 200 pp. each. Plates. Munich, 1832. Jos. Lindaur.

WOLFRAM, LUDWIG FRIEDRICH. "Lehre von der Erdarbeiten beim wasserbau." Text, 200 pp. Plates. Vienna, 1837. Carl Gerold.

Description of river, harbor, and canal work.

SGANZIN, M. J. "Cours de constructions." Atlas 15 × 24 × 2, 1839–1841.

An extensive collection of designs for river and marine work, including foundations and bridges.

MISSARD, M. "Cours de construction—La navigation des rivières et des canaux." Text 8 × 12, 400 pp., 65 plates 12 × 18. Paris, 1841. C. Goeury et V. Dalmont.

A course of lectures in the Ecole de Ponts et Chaussées.

MARY, M. "Cours de navigation intérieure." Text 12 × 15, 150 pp. Cuts and plates. Paris, 1850. Ecole Nationale des Ponts et Chaussées.

Lectures delivered in the Ecole Nationale.

BECKER, M. "Der wasserbau." Text 6 × 10, 300 pp. Atlas. Stuttgart, 1856. Carl Mäcken.

A general treatise with full description of the Danube chain navigation.

BEARDMORE, NATHANIEL. "Manual of Hydrology." Text 5 × 10, 400 pp. Tables. London, 1862. Waterloo & Sons.

Hydraulic and other tables, rainfall and evaporation, rivers and flow of water, tides, estuaries, and tidal rivers.

Chiolich-Löwensberg, Herman von. "Anleitung zum wasserbau." Text 12 × 15, 300 pp., 2 Vols. Plates. Stuttgart, 1864. C. Hoffmann.

A general treatise with an extended callection of plates on river work, foundations and docks.

Bauernfeind, Carl M. "Wasserbaukunde." Text 15 × 20, 30 double-page cuts. Munich, 1866. J. G. Cotta.

River and stream correction.

Hagen, G. "Handbuch der wasserbaukunst." 6 Vols., Text 5 × 8, 500 pp. each; 6 Vols., plates, 20 × 12. Berlin. Ernst & Korn, 1864 to 1869. Vol. 1, Springs, water supply and foundations; Vol. 2, Bank protection, river works and navigable canals; Vol. 3, Correction of streams; Vol. 4, Canal gates, ship canals and dykes; Vols. 5 and 6, Sea, bank and harbor works.

A standard German encyclopedia.

Lagrené, H. de. "Cours de navigation intérieure. Fleuves et rivières." Two Vols., Text 12 × 15, 250 pp. and 450 pp. resp., 2 Atlases, 12 × 15. Paris, 1869–1873. Dunod.

River canalization. Detailed description of the Chanoine and Poiree weirs. An excellent work.

Stevenson, D. "Canal and river engineering." One Vol., 6 × 10, 350 pp., 2d ed. Edinburgh, 1872. Adam Black.

Debauve, A. "Navigation fluviale et maritime." Paris, 1874. Dunod.

Maganzini, Italio. "Opere idrauliche." Text 12 × 15, 200 pp. Plates. Florence, 1877. Bencini.

Mountain torrent correction. Description of canals and hydraulic constructions in Holland.

Franzius, L., and Ed. Sonne. "Der wasserbau." 3 Vols., Text 8 × 12, 500 pp. with cuts and plates. Leipzig, 1882. Wilhelm Engelman. Last edition published in 1892–1893.

An encyclopediac work on the whole field of hydraulics. One of the most complete modern publications in this field.

Jackson, L. D. "Hydraulic manual." Text 6 × 8, 500 pp.; 150 pp. tables. London, 1883. Crosby, Lockwood & Co.

Flow in rivers, canals, pipes; hydrology, irrigation, crevasses, etc.

—— "Hydraulic works." Text, 500 pp., 6 × 10. London, 1885. W. Thacker & Co.

Description of hydraulic works in India, Canada, Egypt, and England.

Guillemain, P. "Navigation, rivières et canaux." 2 Vols., Text 6 × 9, 575 pp. each. Cuts. Paris, 1885. Baudry & Co.

An extended discussion of weirs, locks, river correction, etc. An excellent general treatment.

Durand-Claye, Alfred. "Hydraulique agricole et genie rural." 2 Vols., Text 6 × 12, 700 pp. and 500 pp. Many cuts. Paris, 1890. Octave Doin.

A voluminous discussion of modern practice in agricultural engineering.

"Atlas des voies navigables de la France." 2 Vols., plans 12 × 18. Paris, 1894. Imprimerie Nationale.

Details and cross-sections of gates, locks, chutes, etc.

VERNON-HARCOURT, L. F. "Rivers and canals." 2 Vols., Text 6 × 8, 350 pp. Oxford, 1896. Clarendon Press.
Revised edition of a standard English treatise.

INLAND NAVIGATION.

PRIESTLY, JOSEPH. "Navigable rivers, canals and railways in England." Text 10 × 12, 800. London, 1831. Longman, Rees, Orme, Brown & Green.
A descriptive treatise giving names of engineers and acts of Parliament affecting public works.

GAUTHEY. "Memoires sur les canaux des navigation." Text 10 × 12, 400 pp., with plates. Liége, 1843. J. Ledoux.
A description of the Canal du Charolais.

GAMOND, THOMÉ DE. "Memoire sur le régime général des eaux courantes." Pamphlet in 8°. Paris, 1871. Dunod.
Scheme of inland navigation for France.

BADOIS. "Amélioration des voies navigables. Études à propos de l'ouvrage de M. Malézieux sur les travaux publics des Etats-Unis." Pamphlet in 8°. Paris, 1875. Baudry.

WEBER, MAX MARIA. "Die wasserstrassen Nord-Europa's." Text 8 × 12, 400 pp. Leipzig, 1881. Wilhelm Engelman.
Valuable comparative statistics of European canals.

LANFRANCONI, ENEA. "Über die wasserstrassen Mittel-Europa's und die wichtigkeit du regulirung des Donau stromes." Text, Atlas, 24 × 30. Maps and profiles. Vienna, 1881. W. Heinrich.

BEEKMAN, A. A. "Nederland als Polderland." Text 8 × 12, 500 pp. Several maps. Zutfen, 1884. W. J. Thieme & C^ie^.
Canals, dykes and interior navigation in Holland.

"Mémoires du Congrès International de navigation interieure." Text 8 × 12, 2 Vols. Brussels, 1885. Imprimerie H. Luppens Père.

JEANS, J. STEPHENS. "Waterways and water transport." Text 6 × 8, 500 pp., with illustrations. London, 1890. E. Spon.
A popular treatise on general transportation problems with special reference to the Panama, Suez, Manchester, and Nicaragua canals.

"Fourth international congress on inland navigation held at Manchester." One Vol. in folio. Manchester, 1890.

VETILLART, H. "La navigation aux Etats-Unis." Text 12 × 15, 550 pp., with plans. Paris, 1892. Imprimerie Nationale.
A valuable survey, historical, financial and descriptive of the inland and coast navigation works of the United States.

"Papers of the Sixth International Inland Navigation Congress. The Hague, 1894.

LEACH, SMITH S. "Inland navigation in the United States." Proc. Inst. C. E., 1897, p. 247.
General description.

TRANSACTIONS

OF THE

ASSOCIATION

OF

CIVIL ENGINEERS

OF

CORNELL UNIVERSITY

VOL. VII 1898–1899

CONTAINING

ADDRESSES BY NON-RESIDENT LECTURERS, MISCELLANEOUS
PAPERS, CONSTITUTION, AND LIST OF MEMBERS
OF THE ASSOCIATION

NOTE.—This Association is not responsible for any statements or opinions advanced in any of its publications.

ITHACA, NEW YORK
MAY, 1899

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TRANSACTIONS OF THE

ASSOCIATION OF CIVIL ENGINEERS,

CORNELL UNIVERSITY.

VOL. VII, - - - - - MAY, 1899.

Copies of this and preceding issues can be obtained from Professor C. L. Crandall, Corresponding Secretary of the Association of Civil Engineers of Cornell University, Lincoln Hall, Ithaca, N. Y.

Editor, - - - - - - E. E. LANPHER
Managing Editor, - - - - WAGER FISHER

Assistant Editors.
C. H. BELDEN, A. B. LUEDER, W. O. WHITE

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CONSTITUTION

OF THE

ASSOCIATION OF CIVIL ENGINEERS

OF CORNELL UNIVERSITY.

PREAMBLE.

We, the undersigned, members of the Senior and Junior classes in the College
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ERRATA.

Page 54, for 379 read 3.79.
Page 61, for allignment read alignment.

; of

2. All Alumni of this college and all students recognized as upperclassmen, and registered in the College of Civil Engineering, are eligible to membership in this Association.

3. Any eligible person may become an honorary member by a two-thirds vote of the members present at any regular meeting. Such members shall have privileges of active members except those of voting and holding office, and shall be exempt from all dues.

4. The membership fees for this Association for all active graduate members shall be $1.00 per annum. All money received from such membership fees shall be devoted to defraying cost of publication of non-resident lectures delivered before the Association. All other expenses of this Association shall be met by direct tax upon the undergraduate members.

5. A copy of each lecture delivered before this Asseciation shall be forwarded to each member of the Association.

ARTICLE III.

OFFICERS.

1. The officers of the Association shall consist of a President, Vice-President, Recording Secretary, Corresponding Secretary, and Treasurer.

CONSTITUTION

OF THE

ASSOCIATION OF CIVIL ENGINEERS

OF CORNELL UNIVERSITY.

PREAMBLE.

We, the undersigned, members of the Senior and Junior classes in the College of Civil Engineering of Cornell University, do hereby form ourselves into an Association for the discussion of engineering topics, and the promotion of general information on engineering subjects, and do hereby agree to abide by, and sustain the following Constitution and By Laws :

ARTICLE I.

NAME.

1. This Association shall be known as the Association of Civil Engineers of Cornell University.

ARTICLE II.

MEMBERSHIP.

1. The Association shall consist of Active and Honorary members.

2. All Alumni of this college and all students recognized as upperclassmen, and registered in the College of Civil Engineering, are eligible to membership in this Association.

3. Any eligible person may become an honorary member by a two-thirds vote of the members present at any regular meeting. Such members shall have privileges of active members except those of voting and holding office, and shall be exempt from all dues.

4. The membership fees for this Association for all active graduate members shall be $1.00 per annum. All money received from such membership fees shall be devoted to defraying cost of publication of non-resident lectures delivered before the Association. All other expenses of this Association shall be met by direct tax upon the undergraduate members.

5. A copy of each lecture delivered before this Asseciation shall be forwarded to each member of the Association.

ARTICLE III.

OFFICERS.

1. The officers of the Association shall consist of a President, Vice-President, Recording Secretary, Corresponding Secretary, and Treasurer.

2. The President shall preside at all meetings of the Association and enforce the Constitution and By-Laws, and shall call special meetings at the request of five active members.

3. The Vice-President shall take the chair at the request of the President, and shall act as President in his absence. The Vice-President shall be chairman of the appointment committee.

4. The Recording Secretary shall keep minutes of the proceedings of all meetings of the Association and shall post notices for the same.

5. The Corresponding Secretary shall attend to all the necessary correspondence of the Association. He shall be elected from the Faculty of the college.

6. The Treasurer shall receive all money and dues, and shall pay all bills of the Association, such bills to meet the approval of the Executive Committee before such payments. He shall make a report when called upon by the Association and also when his term of office expires. He shall be chairman of the Executive Committee.

7. The officers shall be chosen by ballot at the last regular meeting of the spring term, from the Junior Class, and shall hold office until their successors are elected.

8. All vacancies shall be filled by a special election—only members of the Senior Class being eligible to hold office.

ARTICLE IV.

COMMITTEES.

1. There shall be three Standing Committees, an Executive Committee, a Committee on Appointments, each consisting of three members, and a Publication Committee, consisting of five members. These committees shall be appointed by the President at the beginning of the year.

2. The Executive Committee shall see that the rooms of the Association are ready for occupancy previous to all meetings, and shall transact such business as may be referred to it by the Association.

3. The Committee on Appointments shall make appointments for all literary exercises for each meeting, and such appointments shall be posted at least two days before the meeting. The committee shall furnish the Secretary with a list of such appointments.

4. The Publication Committee shall have charge of the compiling, publication, and sale of the Transactions.

ARTICLE V.

AMENDMENTS.

The Constitution or By-Laws may be amended by a two-thirds vote of all members present at any regular meeting; such amendment to be before the Association at least one week.

AMENDMENT I.

The initiation fee for membership to this Society shall be one dollar, and this fee shall cover all dues for the first year of membership. For each succeeding year, an annual due of one dollar shall be charged. The annual membership fee shall entitle the member to a copy of the Transactions published during each year of membership.

BY-LAWS.

ARTICLE I.

REGULAR MEETINGS.

Regular meetings shall be held once in two weeks, in the Association rooms, commencing on the first Friday after registration week, and ending on the last Friday but one before examination week of each term.

ARTICLE II.

QUORUM.

One-third of the active undergraduate members of the Association shall constitute a quorum. No business can be transacted without a quorum being present.

ARTICLE III.

ORDER OF PROCEEDINGS AT A REGULAR MEETING.

1. Roll call.
2. Minutes of Preceding Meeting.
3. Literary Exercises.
4. Unfinished Business.
    a. Report of Standing Committees.
    b. Report of Special Committees.
    c. Report of Officers.
    d. Miscellaneous Business.
5. New Business.
6. Adjournment.

ARTICLE IV.

EXERCISES.

The exercises shall consist of discussions, memoirs, essays, papers, lectures, and such other exercises as the Association shall from time to time direct.

ARTICLE V.

SUSPENSION OF BY-LAWS.

A By-Law may be suspended for one meeting by a vote of two-thirds of the members present.

H. R. LORDLY,
E. J. FORT,
H. D. ALEXANDER,
Committee.

OFFICERS FOR 1898-99.

President,

FRIEND P. WILLIAMS.

Vice-President,

LESLIE McHARG.

Treasurer,

R. C. TAYLOR.

Corresponding Secretary,

PROFESSOR CHAS. L. CRANDALL.

Recording Secretary,

A. H. HIGLEY.

Appointment Committee,

LESLIE McHARG, *Chairman*, ALEXANDER THOMPSON, JR.,
OGDEN MERRILL.

Executive Committee,

R. C. TAYLOR, *Chairman*, M. A. RUE,
C. L. BARTON.

Publication Committee,

WAGER FISHER, *Chairman*, E. E. LANPHER,
C. H. BELDEN, A. B. LUEDER,
W. O. WHITE.

PRESIDENT'S ADDRESS.

To Members of the Association of
Civil Engineers of Cornell University:

According to the usual custom, it becomes my privilege and duty, as retiring President, to address you on matters of interest to the Association during the past year, and, if possible, to suggest lines of improvement, which will further the purposes for which the Association was organized.

The meetings of the Association have been held every two weeks throughout the year.. An especial feature this year, has been the lectures by noted non-resident engineers; including Major Symonds, Messrs. Trautwine, Morrison, Bates, Goldmark, Rafter, Molitor, Duryea and Weston. The Association has been very fortunate in being able to listen to such a large number of eminent engineers. The value of lectures given by such practical business men, and men of such great ability in their branches of engineering, cannot be over estimated. Nearly all of those lectures were held in the Physical Lecture Room, Franklin Hall, where an excellent opportunity is afforded the lecturer for illustrating his lecture with lantern views.

Besides the non-resident lecturers, the Association has had the pleasure and benefit of listening to several members of the Faculty and members of the Association. Among the most important of these lectures, was one by Professor Church, on "Different Hydraulic Laboratories throughout the Country"; one by Professor Crandall, upon "The Failure of a Whipple Truss; one by Professor Ogden, upon "The Field of Work of the Sanitary Engineer"; and one by Professor Jacoby, on "How to Make a Book." On behalf of the Association, I wish to express its hearty appreciation of the kindness of all those who have so interested themselves in engineering study, and lectured before the associasion during the past year.

The Transactions, now in its seventh volume, has been enlarged and improved since its earlier issues. Such excellent lectures as are delivered before the Association, and such papers as are obtainable from time to time from alumni members, furnish material of a most valuable and interesting character for this annual publication. It is sincerely hoped that the Transactions will always hold a high standard in engineering literature, and that all members will take part in advancing its interests and helping it to always attain a high standard of excellence. The Publication Committee, this year, has worked long and earnestly in getting out the Transactions and deserves a great measure of credit for its untiring efforts.

Turning to the financial side of the Association, I am sorry to note that the Association's indebtedness on account of the Transactions, amounted to nearly two hundred dollars at the beginning of the year. As Professor Crandall, with his usual generous interest in Association matters voluntarily assumed this large debt of the association, I think it is important to ask both alumni and active members for their hearty support in assisting the Association out of debt. A portion of this indebtedness, however, had been paid off during the year; through the kind responses from alumni and by a tax levied on active members.

There is no better suggestion that I have to offer, than the oft repeated one urging members to attend meetings regularly, and to take an active interest in all the affairs of the Association. Our Director, Prof. Fuertes, in a recent lecture, pointed out the necessity of *seeing* things, not *looking* at them. Hand in hand with the ability to see things, goes the ability to tell about them. I would like to suggest, therefore, that the members take advantage of the excellent opportunities afforded by the Association for preparing and discussing engineering subjects.

As the Seniors leave the Association now, and its care falls upon the Juniors, I would like to suggest that the Juniors make special efforts to get all the Sophomores into the Association as soon as possible, that meetings, early in the year, may be largely attended; and that an invitation always be extended to underclassmen to attend meetings. I would like to urge that the Appointment Committee make out program of meetings as far ahead as possible; in order that plenty of time may be given for preparation, and regularity of meetings may be insured. The Current Events Committee has made some interesting reports during the year. In this connection, I would like to suggest that the membership of the committee be changed from term to term, and that opportunity be given for discussing the subject brought up, if possible.

I would like, now, to thank the members of the Association for their cordial support during the past year, and the officers and committees for their kind co-operation in Association matters.

The Members of the Class of Ninety-nine now bidding farewell as active members in the Association, I am sure, carry with them pleasant memories of their connection with the Association and will always be interested in its welfare. I now take pleasure in presenting the chair to the President for 1900.

FRIEND PITTS WILLIAMS.

NON-RESIDENT LECTURERS,

1898-1899.

*Major Thomas W. Symonds—Jan. 20. The Corps of Engineers of the United States Army and Some of its Works.

John C. Trautwine, Jr.,—Feb. 3. The Waterworks of Philadelphia.

Onward Bates—Feb. 17. The Engineer at Work.

*Henry Goldmark—March 17. Locks and Lock-Gates for Ship Canals.

*George W. Rafter—April 14. Data of Stream Flow in Relation to Forests.

*David Molitor—April 28. The Present Status of Engineering Knowledge Respecting Masonry Construction.

Edwin Duryea, Jr., '83—May 5. The Qualities and Habits of Work Most Necessary to Secure Success in Civil Engineering. An Address to the Graduating Class.

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* Contained in this volume.

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Greene, Almon C.	*C.E.*, '75;	Palmyra, N. Y.,	July 28, 1897
Gunner, Daniel W.	*C.E.*, '87;	Schagticoke, N. Y.,	Oct. 10, 1887
Holbrook, Ernest M.	*C.E.*, '89;	*M.C.E.*, '90; Ithaca, N. Y.,	Oct. 9, 1892
Hulse, Howard C.	*C.E.*, '91;	Brooklyn, N. Y.,	Feb. 20, 1893
Landers, Herbert H.	*C.E.*, '90;	Green Island, N. Y.,	Feb. 4, 1893
Lyman, George F.	*C.E.*, '73;	Tenafly, N. J.,	Dec. 25, 1880
MacMullen, Justus C.	*C.E.*, '76;	Unionville, N. Y.,	Jan. 31, 1888
Preston, Kolce	*C.E.*, '73;	Wilmington, Del.,	Jan. 4, 1876
Rogers, Jesse A.	*C.E.*, '91;	Evans Mills, N. Y.,	April 24, 1897
Sheldon, Daniel C.	*C.E.*, '83;	Delphi, N. Y.,	Oct. 2, 1893
Shepard, Frank W.	*C.E.*, '86;	Medina, O.,	Feb. 10, 1892
Smith, George La T.	*C.E.*, '71;	*M.C.E.*, '74; Canandaigua, N.Y.,	June 25, 1892
Smith, William J.	*C.E.*, '79;	Charleston, N. Y.,	Dec. 3, 1886
Stewart, Neil, Jr.	*C.E.*, '87;	York, N. Y.,	March 30, 1891
Tilley, George A.	*C.E.*, '73;	Washington, D. C.,	March 14, 1877
Tompkins, John H.	*C.E.*, '73;	Poughkeepsie, N. Y.,	July, 1879
Van Cauteren, Emile A.	*C.E.*, 97;	Pringy, Seine-et-Marne, France,	July 4, 189[illegible]
Viegas-Muniz, Joaquim	*C.E.*, '77;	Piracicaba, Brazil,	About 1883
Wightman, Willard H.	*C.E.*, '81;	Ashland, Ore.,	Oct. 29, 1889

In Memoriam.

EMILE ALFRED VAN CAUTEREN, C. E. '97.

The death of Mr. Emile Alfred Van Cauteren occurred July 4, 1898, he being one of the victims of the sinking of the French Liner, La Bourgoyne.

Mr. Van Cauteren resigned his position with the United States Deep Water-ways Commission July 1st, in order to accompany his sister to their home at Pringy, France. They sailed from New York July 2nd, and in the early morning of July 4th the Bourgyne collided with the British ship Cromartyshire off Sable Island. Mr. Van Cauteren's name was not among the list of survivors.

As members of the Association, we all feel his death deeply, but to those who were immediately associated with him in the University, his death appeals most strongly, as we lose not only an associate and acquaintance, but a friend faithful and true.

Mr. Van Cauteren was born September 14, 1875, at Brooklyn, New York. In June, 1891, he graduated from the Boys' High School, Brooklyn, and after a year's further study entered Cornell, registering in Civil Engineering with the class of 1897. On entering the University he won the Stewart L. Woodford Scholarship, and by his thorough systematic work, soon placed himself at the head of his class.

After graduating in June, 1897, Mr. Van Cauteren spent the summer at his home in Pringy, France. He returned in the fall and entered the service of Shaler and Crimmins as inspector on the New York subways. This position he resigned in November to accept a position under the U. S. D. W. Survey on the Western Division of the Oswego Route, where he remained until the time of his resignation in July to return to France.

THE CORPS OF ENGINEERS OF THE UNITED STATES ARMY AND SOME OF ITS WORKS.

MAJOR THOMAS W. SYMONDS, CORPS OF ENGINEERS U. S. ARMY.

There is one branch of engineering that I did not see mentioned in Professor Fuertes' engineering list at the head of his letter to me ; it is the branch to which I have devoted the greater part of my life and about which I am going to talk to you a little while today, the engineering work connected with the improvement of rivers and harbors. As all engineering work is more or less closely allied, I have no doubt that it is often alluded to in your course, however.

While there is much River and Harbor engineering work carried on by private individuals, corporations and municipalities in the United States, yet the Government itself has taken general charge of all the navigable waters, rivers and harbors of the country, and has undertaken the gigantic work of keeping them in order and improving them in the interest of commerce. This work is in the hands of the Corps of Engineers of the Army, to which organization I have the honor to belong, and which is a branch of the War Department. It may seem strange that such essentially peaceful work should be under the Department of War, but it is a very natural outgrowth of the conditions existing in the early years of our country's history.

West Point was established as an engineering school, and for many years it was the leading engineering school of the country, and its graduates were the best fitted to take up the engineering work of the country. Two Corps of Engineers were founded from the graduates, the Corps of Fortification Engineers who were to devote their lives to planning and building the land defenses of the country, and the Topographical Engineers who were to explore and map the vast regions of the then unknown West, lay out routes of communication, build military roads, etc.

It was but natural that when the Government began to make small appropriations for the improvement of a river here and a harbor there, the work should be placed in the hands of one of the Engineer Corps already organized for closely related work. The two corps, Fortification and Topographical, were finally merged into one, the Corps of Engineers as it now exists. An amusing incident connected with this merging was that it was protested against by the then Chief of the Fortification Engineers because the average graduating standing at West Point of the Fortification Engineers was 2½, while the average graduating standing of the Topographical Engineers was 11¼. The celebrated John Phenix, author of the Squibob Papers, and who was an officer of the Topographical Engineers, Capt. Derby, wrote one of his comical and sarcastic letters on the subject ridiculing this protest. He declared that never hereafter could he walk down the street without a feeling that there was a great 11¼ painted on his back as a badge of inferiority.

I have not delved into the records of the past to determine when the first appropriation was made for a work of River and Harbor improvement. The appropriations, small at first, have grown with our growth and with the importance of the commerce of the country, until the aggregate reaches about $300,000,000. The last River and Harbor bill passed in 1896 authorized works and contracts therefor involving about $70,000,000. The responsibility for planning all the works, executing them, and disbursing the money appropriated devolves upon the Corps of Engineers. I may perhaps be pardoned for saying a few words about my corps. It is composed entirely of graduates of West Point and of those who have graduated at the head or very near the head of their classes. When they graduate, they enter the crops as 2nd Lieutenants, and are ordinarily sent to Willets Point to take a post-graduate course in practical engineering and torpedo work. They are then sent as assistants to older officers, and gradually acquire knowledge, wisdom and experience until they are in turn deemed capable of responsible charge of works.

Besides the officers there are a great many civil engineers employed all over the country on these works, and many of the leading engineers of the country have spent some of their earlier years as assistant engineers on River and Harbor work under the War Department. At Buffalo I have had a number of assistants who are graduates of Cornell. I am not going to throw a bouquet at Cornell just because I happen to be here now, but I will say that, judging by the young men who have come to me, a man gets a better practical, general engineering education here than at any other college with which I am familiar.

I would not advise a young man to enter the Government service as a civil engineer, with a view to remaining in it all his life, but a few years service will do any one good, giving him familiarity with the methods of doing work and systems of accountability and responsibility which are the outcome of many years experience, and to which is largely due the success of the Engineer Corps in handling the public works of the country.

A new improvement is only undertaken by the Government after a careful examination. When in the course of time, the people of a particular locality become imbued with the idea that they want some special work of river or harbor improvement undertaken by the Government, they first make their wants known to their representative in Congress, and generally they have no trouble in convincing him of the propriety of the Government's doing the work. The next step is to get an item in the River and Harbor bill providing for a preliminary examination. This preliminary examination is made by the Engineer Officer in charge of works in the vicinity, and it is his duty to decide and report whether or not the project is worthy of being undertaken by the Government, giving his reasons therefor, which must be based upon commercial statistics, the resources of the country, the possible expansion of business, the general physical characteristics and the possibility of the improvement being successful, etc., involving much labor and research and weighing of testimony often unreliable and indirect. If the decision of the local engineer is favorable and it is approved by his Division Engineer and the Chief of Engineers, the next step is for the interested Congressman or Congressmen to get an authority in the next River and Harbor bill for a survey and the preparation of plans and estimates of cost of the improvement. These plans and estimates are when made, submitted to Congress, and Congress then makes an appropriation for the work, or does not make it, as the case may be. Of course, there are exceptions to all rules, and there are works which have been undertaken and carried out which have never been approved by the Engineers, and others which, apparently justifiable at one time, have, by the march of events and new developments, become unjustifiable and of little use.

Besides the work of river and harbor improvement, the building and repairs of the light-houses all along our coasts come under the Corps of Engineers. There are about 2,000 of these light-houses, and some of them have been engineering works of the greatest difficulty.

Planning and building the fortifications for the defense of the country against a foreign foe also is in the hands of the Corps of Engineers of the Army, and this work has for several years been progressing with great rapidity. So much for the Corps of Engineers and its general duties.

It is natural that we should like to tell about the big and successful works with which we are familiar, and so I will ask you to go with me in imagination across the continent to the coast of Oregon and Washington where I was stationed for many years. The largest, most important and most successful engineering work which has been done on the Pacific coast is the improvement of the entrance to the Columbia River from the ocean.

But little over one hundred years ago, 1792, the great river of the Northwest was first discovered and entered by Capt. Gray in the good ship "Columbia," from which ship the river takes its name. Gradually the country drained by the Columbia settled up, passing through the hands of the fur traders, the gold hunters, and the lumbermen, to the agriculturists who now control it, and gradually it's commerce increased and the number of ships coming to the river became greater and greater. But for many years the entrance to the Columbia River and the Columbia River bar were the terror of mariners, many a ship was lost there, and insurance rates were always increased on ships bound therefor. At last the government was induced to take action looking to the improvement of the entrance, and a Board of Engineers was appointed who planned certain works of improvement which have been carried out, and which have proven most successful, and now the Columbia River and bar have lost all their terrors, and the entrance is as safe and easy as the entrance to any bar harbor can be on the stormy Pacific northwest coast.

Inside of the headlands which form its mouth, the Columbia widens out into a great tidal estuary which is some thirty miles long by ten miles broad. Between Fort Stevens on the south of the entrance and Cape Disappointment on the north the distance is six miles. Circling off at a distance of about three to four miles from the entrance is the inevitable ocean bar which is always formed where there are moving sands, and which is the resultant of the confiict that is ever going on between the river currents and ebbing tides on the one hand and the flooding tides and storm waves of Old Ocean on the other.

In its natural condition there was but about 12 to 15 feet of water on this Columbia River bar with deeper channels, sometimes one, sometimes two or three across it. These bar channels were generally deficient in depth to accommodate the ships desirous of entering the river, and were not permanent in position, a most serious fault. Sometimes there was no channel where a fair channel existed but a few months before, and at times, under the influence of storms, the channels changed so rapidly that the pilots themselves could hardly keep track of them Along this portion of the coast violent storms are of great frequency, which of course aggravated matters very much.

The plan adopted for the improvement of the mouth of the river is simplicity itself; it consists of building a single stone jetty 4½ miles long, built out from Fort Stevens and designed to shut off all secondary channels and concentrate the ebbing and flooding tidal currents and the river outflow into one channel properly located and permanent in position. As the work of jetty extension progressed out from Fort Stevens, shutting off the flow to the southward, the channel across the bar became better and better, and when it was finally completed there was a straight channel across the bar over 30 feet deep at low tide and nearly a mile in width, which channel has remained permanent in position and depth ever since. No more successful work was ever done, and its cost was well within the original estimate.

At first it was proposed to do this work by unloading the stone from vessels into the jetty, but this was found to be impracticable, owing to the prevailing rough water. A tramway was therefore built of piles and timber and gradually extended out. The stone brought from the quarries up the Columbia was unloaded at a sheltered wharf on to cars, which were run out on the jetty tramway and dumped. The enrockment was brought up to about 6 feet above high tide to allow for future settlement. For purposes of economy and to prevent the stone sinking too deeply into the sand, brush mattresses were first put down on the bottom. These were about 5 feet thick, of stout brush well bound together. There were many interesting details about the work which time will not permit me to go into here. It was quite a novel experience enjoyed by a great many people, of getting on a locomotive and riding on a tramway 4 miles and more out to sea directly towards China. Sometimes when there was a storm the experience was rather fearsome.

The total cost of the Columbia River jetty work was just about $2,000,000.

On the Pacific Coast are many other works similar to this, though not of as great magnitude, and other interesting works of different character, but I can not mention them now, and will ask you to do as I did in 1895, jump from the Pacific Coast to the Great Lakes.

THE GREAT LAKES.

There has been built up on the Great Lakes a commerce that is enormous, almost inconceivable to those not brought into contact with it in a business or statistical way.

Looking back but a few years, we see on the lakes only the canoes of the Indians, then came the larger bateaux of the fur traders, then the small sailing vessels of the traders, explorers, and settlers, then larger sailing vessels, the advent of steam, and then steel vessels which have increased in size and carrying capacity and economy of operation, until now the business of the lakes is the model and one of the wonders of the world.

A strictly up-to-date, modern lake freight vessel is about 430 feet long, 50 feet broad, and 30 feet deep. It draws about 18 feet of water and carries 7000 tons of freight, or, if its cargo is grain, about 250,000 bushels, or the full crop of 100 grain farms of 160 acres each.

About 80 per cent of the tonnage of the lakes is made up of the three items of coal, iron ore and grain, about equal proportions of each. Anthracite coal is mostly shipped from Buffalo, and soft or bituminous coal from Ohio and Pennsylvania Lake Erie ports.

It may interest you to know that in shipping soft coal, it is taken a whole car load at a time and dumped into the ship by means of great car-dumping machines. The expense of operating these machines is less than a cent a ton of the coal handled. The iron ore is brought from Lake Superior ports and mostly goes to Cleveland, Ashtabula, Conneaut, Erie, and other ports, and is then largely shipped by rail to that great center of the iron and steel industry, Pittsburg.

Most of the grain products of the Northwest are shipped from Chicago, Duluth, Milwaukee, and other up-lake ports to Buffalo, where it is unloaded into the great elevators, and thence shipped by rail or canal to the seaboard, or to the mining, manufacturing and commercial centers along the Atlantic sea coast.

There is also a large amount of lumber transported on the lakes, but this is declining as the forests are cut off, while the other items mentioned are increasing.

There is no feature of lake commerce more interesting and marvelous than the appliances and system for unloading and loading vessels in port. It is not uncommon for a vessel to arrive at a lower lake port with 4,000 to 5,000 tons of iron ore or grain aboard, and to have this all taken out and as much coal put aboard, and the vessel to be off on her trip, all within twenty-four hours. Such quick dispatch prevails nowhere else in the world and largely accounts for the exceedingly low rates at which freight is carried, the steamers wasting no time in port, but keeping constantly on the go.

We are all naturally accustomed to look upon the Suez Canal which connects the teeming millions of Europe with the other teeming millions of Asia as the great throat of commerce of the world. In the general world commerce, it is the most important *link*. And yet the commerce passing through the Suez Canal is much less than one-half that passing through the St. Mary's Canal connecting Lake Superior with Lake Huron, and only about one-fourth that passing through the Detroit River connecting Lake Erie with the upper lakes. And all this notwithstanding the fact that the lakes are closed by ice four to five months each year.

But this enormous commerce carried through the lakes in the magnificent modern lake vessels was not always possible. The Government had to do a vast amount of work and expend large sums to render it possible.

There are two principal items of work in improving the commercial facilities of the lakes in which the Government has been concerned ; to improve and increase the navigable capacity of the channels connecting the lakes and to supply and improve the harbors for the transaction of business. It is the aim of the Government to give a channel 20 to 21 feet in depth from the foot of Lake Erie to the head of Lakes Superior and Michigan, and the work necessary to accomplish this has been nearly completed.

This has involved a great deal of work in dredging and rock excavation in the Detroit River, the dredging of a canal through the flats of Lake St. Clair, channel deepening at the foot of Lake Huron, and above all, the improvement of the St. Mary's River, the outlet of Lake Superior, by a vast amount of dredging and rock work, and the construction of locks to overcome the fall at the Sault Ste. Marie, or falls of the St. Mary's River. Without counting the locks at the Sault Ste. Marie, the total amount that has been expended by the Government in deepening and widening these interlake channels is about $6,000,000.

Just at the foot of Lake Superior their is a fall in its outlet, the St. Mary's River, of eighteen feet. The first lock built to overcome this fall was constructed by the state of Michigan in 1855, but it was small and commerce soon outgrew it. The general government then in 1870 built a lock which it was believed would be sufficient for all time, but so rapid was the growth of commerce that in 1896 another lock was completed which is one of the largest in the world. It is 100 feet wide and 800 feet long and capable of taking four of the ordinary lake vessels at one lockage, which, in fact, it frequently does. The Canadians have also built a lock 80 feet wide and 1000 feet long at the Sault, so that now there are three great locks there practically side by side. The last lock built by the United States cost nearly $4,000,000. I said that the first lock was built at the Sault in 1855, but this is really not so.

Many, many years ago, just when I do not know, some of the old fur traders or voyagers built there a lock to pass their bateaux up and down. On a recent visit to the Sault I found that this had been unearthed and partially reconstructed, and it serves as an object lesson of the growth of lake commerce. This little old lock was about 8 feet wide and 25 feet long. It would take 400 of these little locks to equal in size the new lock just completed, or about 1200 of them to equal the three locks now in service and in continual operation.

Most of the harbors on the Great Lakes which are the centers of great commercial activity are to a large extent artificial, and many are now inadequate to the demands upon them. Buffalo is quite a typical lake harbor; it has a decided similarity to the harbors at Chicago, Cleveland, Milwaukee and other lesser harbors, such as Ashtabula, Conneaut, etc., and a description of it may, to a reasonable extent, be considered a description of all.

The harbor of Buffalo is a small stream, Buffalo Creek, which in its natural condition was hardly big enough at times for boys to swim in. There were, however, some deep holes near its mouth which were able to accommodate the little light-draft sailing vessels of seventy-five or eighty years ago. These coming to the creek developed business, and as the business increased and the vessels grew larger, the enterprise of the inhabitants kept pace with the march of events and the creek was dredged out, and this has been kept up up to the present time.

The city has spent a very large amount of money in widening and deepening the little waterway, and when this was found not to give sufficient room, the city excavated a ship canal between the creek and the lake for the further accommodation of vessels, and this has been further supplemented by individuals and corporations.

In the march of events the General Government was called upon for aid, and extended it by building the piers at the mouth of the creek intended to insure the stability of the entrance and protect vessels in entering. Later on, it built the existing breakwater 1½ miles long to further protect the entrance to the harbor, and to give a sheltered anchorage and harbor of refuge protected from the storms and waves of the open lake.

This was the condition of the harbor when I came to Buffalo in 1895. But commercial developments have for several years been going on with great rapidity, and with the advent of the great lake steamers, their constantly increasing size and numbers and growing business, it became plain to many thoughtful men that Buffalo Creek and the City Ship Canal were not adequate or suited to the future growth and development of Buffalo. The movement was started to get the Government to build an outer harbor in the lake where business could be transacted free from the annoying complications attending the use of the narrow creek and canal by the large vessels. To this end, it was planned to extend the old breakwater to the southward about 2½ miles to a jutting point of land called Stony Point, thus shutting in an area of the lake about four miles long and ⅔ of a mile wide. The water behind this breakwater will be thoroughly protected, and it is intended that the upland owners shall have the right to build piers and wharves out to about half way from the shore to the structure, which wharves, piers, etc., can all be readily reached by rail, as well as by water. This work, it was estimated, would cost $2,200,000, and the Government authorized it and has provided funds for it as fast as they can be profitably expended, and for two years the work has been going on.

When it was decided that it was advisable to extend the breakwater to Stony Point, the first thing necessary was a survey to determine the depth of the water and the character of the bottom. This survey was made through the ice, and developed the fact that along the projected line the water was on the average about 30 feet deep, and that the bottom was mud to an average depth of about 30 feet more, and that underlying the mud was a bed of rock. In some places the mud was 60 feet deep, making a total depth to rock of 90 feet of water and mud.

The geologists say that this rock bottom developed by borings shows the bed of an old river which existed long before there was any Niagara Falls, which river flowed from east to west, and is crossed almost perpendicularly by the line of the breakwater.

Now, a breakwater interposed broadside against the storms and storm waves of Lake Erie, is subjected to enormous strains and requires a good foundation which this muddy bottom would not give for the ordinary type of timber and stone breakwater that has been almost exclusively built on the lakes. Somehow or other the foundation must be carried down to the rock at the bottom of the mud, and it was determined to do this by digging an enormous ditch in the bottom of the lake and then filling this ditch with gravel, surmounting this with stone, and then mounting upon this the timber and stone breakwater extending from a depth of 22 feet to a height out of water of 12 feet. For a mile and a half the work is to be of this character. The trench will have a maximum depth of nearly a hundred feet, and a bottom width of about 60 feet. It is being excavated by a giant dredge made expressly for it, and it is being back-filled with gravel dredged from the Niagara River. This trench and its filling are under the immediate charge of a Cornell graduate, Mr. Norris M. Works. The timber crib is 36 feet wide and is strongly framed together of 12-inch square timbers, and all compactly filled with stone and decked over.

A study of the borings revealed the fact that on the northern portion of the old prehistoric river, the mud contained a stratum of sand, and on this account and for other reasons, it was concluded to build the northern portion of the new breakwater, a mile in length, of stone without using any timber. This we call the Rubble Mound Breakwater. Its cross-section as it is being built is, as nearly as practicable, the cross-section that would be developed in a random row of large rubble stone dumped into the lake and subjected to storm action. It is founded directly upon the mud bottom, and has a width of 140 feet at the base, which insures it against overturning. If this rubble mound had been built entirely of quarried stone, its cost would have been greater than the appropriation would stand, so that the expedient was adopted of using a hearting of gravel. The gravel costs 13 cents per cubic yard, while the stone costs about $1.25 per cubic yard, and as about half the total bulk is gravel, the saving amounted to over half a million dollars. As the structure is being built the gravel is encased in the ordinary rough quarry rubble stone, and the whole is capped with large stones carefully shaped and carefully and closely laid in position by derricks. These capping stones average about 8 tons weight each and extend down to a depth of 15 feet on the lake side and 10 feet on the harbor side, and to the height of 12 feet above the water.

The placing of this capping stone is very particular and careful work, and it is under the charge of two Cornell graduates, Mr. C. M. Emmons and Mr. Lynn L. Davis.

It will require about three years more to complete this great work, which, when completed, will be the longest breakwater in the world. In fact, with the old and new breakwaters together, the Buffalo breakwater as a whole will be fifty per cent. longer than any other in the world. It is unique in several particulars, notably in the great foundation trench and its filling, the use of a gravel hearting for the Rubble Mound, and the care and accuracy with which the capping stones of the Rubble Mound are placed.

There will be an opening of 600 feet width at a distance of 2,800 feet from Stony Point, the free ends being placed en echelon to better guard against the entrance of storm waves.

When this work is completed Buffalo will have a harbor second to none on earth, and one where ships can meet railroad cars and canal boats and exchange loads at the very minimum of expense.

NOTES ON THE USE OF CONCRETE IN DAMS, WITH A DESCRIPTION OF THE COLD SPRING DAM.

NOAH CUMMINGS, C.E., '94, ASS'T ENG'R BOARD OF PUBLIC IMPROVEMENTS, NEW YORK.

The employment of concrete in masonry construction has gradually extended during the past few years from its use in core walls, in foundations and as back-filling for rubble and cut stone facing to its use in jetties, breakwaters, graving docks, canal locks, dams and reservoirs. In these latter works, it is more or less directly exposed to the action of the sun, frost and water. It has been successfully used in several important locks built under the supervision of the U. S. Engineer Corps, among which are those on the Cascade canal in Oregon, the Illinois and Miss. canal in Illinois, the Coosa River in Alabama and the Osage River in Missouri. Among the more important dams built of concrete are the San Mateo dam of California, 170′ high; the Beetaloo dam in South Australia, 118′ high; the Periyar dam in India, 177′ high; the La Grange dam in California, 127′ high, and the Butte City dam in Montana, 120′ high. The last three of these have rubble masonry faces and the last two have throughout them boulders embedded in the concrete.

The advantages in the use of concrete are cheapness, adaptability, rapidity in construction and ease in making repairs. The items that make up the cost of rubble masonry are cost of stone at quarry, cost of transportation to the site, cost of cement and sand and cost of laying. These items for concrete are cost of rock at quarry, cost of crushing, cost of transportation, cost of cement and sand, and cost of making.

Since rubble stone when laid has only 30 to 35 per cent. voids as compared with crushed stone, which has 45 to 50 per cent., rubble masonry will require only 65 to 70 per cent. as much cement and sand as concrete. In rubble masonry, it is also customary except near the foundation and in the facing, to use American cement, but in concrete for impervious work, Portland cement has almost universally been used. The latter costs at the cement works, approximately 100 per cent. more than the former, but this percentage will diminish with the increase in the distance from the factory. However, since American cement weighs one-fourth less per barrel than Portland cement, it will have lighter freight charges and this percentage will not diminish so rapidly as it would if the cost of transportation for the two cements were equal. Rubble work has, furthermore, no cost item for crushing, the amount of which varies from 10 to 25 cents per cubic yard of concrete depending on the size of the stone as it blasts out of the quarry, the size to which it is crushed and the facilities of the crushing plant.

The cost of laying rubble varies with the facilities of the plant employed, the size and shape of the stone and the form of the structure. The cost of making concrete varies with the facilities of the plant employed and the shape of and cost of materials for the forms. Force accounts for these two items are given below, it being assumed therein that the costs for superintendence, coal and rigging of plant, which are not included, are the same for both. The unit cost of labor is taken at $1.50. Owing to the great amount of extra labor required which cannot be calculated in advance, force accounts should not be used in estimating the probable cost of a given work unless one is able from experience in works with similar conditions, to add the proper percentage for these extras. The following force accounts are given for the purposes of comparison only :

Force required for one derrick in laying ruble.

1 Foreman, at $4.00	$ 4.00
5 Masons, at $3.50	17.50
2 Tagmen, at $1.75	3.50
1 Helper for masons handling mortar, at $1.50	1.50
2 Men mixing mortar, at $1.50	3.00
1 Hoisting engineer, at $2.00	2.00
Total	$31.50

This force in a 10-hour day with stone ½ to 1 cu. yd. in size, will lay 35 cu. yds., which gives a cost of $0.90 per cu. yd.

Force required for one derrick in mixing and placing hand-mixed concrete.

Mixing cement	2 men	
Mixing concrete	7 men	
Ramming concrete	3 men	
	12 men at $1.50,	$18.00
1 Carpenter building forms, at $2.00		2.00
1 Helper, at $1.50		1.50
1 Hoisting engineer, at $2.00		2.00
2 Tagmen, at $1.50		3.00
1 Foreman, at $3.00		3.00
Total		$29.50

This force in a day of 10 hours will mix and place 50 batches of 18 cubic feet each when rammed in place, or 33⅓ cubic yards in all, which gives a cost of $0.88 per cubic yard. To this must be added from 5 to 10 cents per cubic yard of concrete for cost of lumber for forms.

If the number of cubic yards of masonry is sufficient to warrant the installation of a mixer for the concrete, this cost item for mixing and placing may be greatly reduced, as shown by the following force account.

Force required for mixing and placing machine-mixed concrete.

Handling cement from shed	3 men	
Loading cement and sand in buckets,	3 men	
Loading broken stone in buckets	8 men	
Tending mixer and cars underneath,	6 men	
Spreading and placing concrete	7 men	
	27 men, at $1.50,	$40.50
2 Hoisting engineers, at $2.00		4.00
1 Engineer for mixer, at $2.00		2.00
4 Tagmen, at $1.50		6.00
3 Carpenters building forms, at $2.00		6.00
3 Helpers, at $1.50		4.50
2 Foremen, at $3.00		6.00
Total		$69.00

This force in a day of 10 hours using a four-foot cubical mixer, will place approximately 140 batches of 1 cubic yard each when rammed in place, or 140 cubic yards in all, which gives a cost of $0.49 per cubic yard, to which, as in the case of hand-mixed concrete, must be added from 5 to 10 cents for cost of lumber for forms.

If the quarry is near the site of the dam so that the cost of transportation is a small item and taking the cost of stone at the quarry constant, the above comparisons of rubble and hand-mixed concrete show that rubble work will be cheaper, as the cost for laying is practically the same in both cases, while the rubble effects a great saving in cost of cement and has not the cost item for crushing. In many regions, however, good rubble stone cannot be found, while in nearly all localities there is plenty of rock that will make satisfactory concrete. When the extra distance that the rubble stone must be transported increases its cost of transportation to a certain point, then the cost of rubble masonry will equal the cost of hand-mixed concrete, and beyond this point the concrete will be cheaper. In the comparison, however, of machine-mixed concrete with rubble, the saving of the former over the latter in cost of laying tends to balance its excess in cost of cement and cost for crushing; and thus the extra distance that the rubble stone may be transported before the costs of the two classes of masonry are equalized, is reduced or perhaps wholly obliterated. As the various items making up the costs for rubble and for concrete depend so much on local circumstances, no complete general comparison can be made. Each special case must be worked out in order to determine which class is the cheaper.

Concrete is better adapted than rubble masonry to those parts of a country where skilled labor and large plants are hard to obtain or not procurable, as it can be built anywhere with native laborers and may be laid with a lighter plant or, as is sometimes done in India and elsewhere, with almost no plant at all. As concrete may be made of gravel, pebbles, shingle or coquina gravel, it is also better adapted to those regions in which any of these is found but where it is difficult to get suitable stone for rubble.

The objections that may be raised to the use of concrete in dams are its permeability or absorption, the shelling off of the surfaces exposed to water and frost, the development of cracks due to temperature changes, and its appearance. These objections may be partially or wholly overcome by employing the methods recommended by various engineers as outlined below.

In reference to permeability, Mr. William Smith gives[1] as the result of a series of experiments, that, to obtain an impermeable concrete, the following conditions must be observed:

1. The cement must be finely ground.
2. The concrete should not be weaker than one cement to six of sand and stone.
3. It should have a sufficient time, at least three months, to set before the application of pressure.

Mr. G. W. Birdsall, Chief Engineer of the Croton Aqueduct, states that concrete of 1 cement, 2 sand and 3 stone or gravel is practically water tight.

Water to a certain extent enters all classes of masonry, and in dams it percolates through the structure in places. Concrete, owing to its many surfaces of adhesion between the stone and the mortar, will be more easily attacked than facing stone masonry, except certain classes of limestones and sandstones which absorb water more freely than concrete. Concrete increases in imperviousness with the increase in the proportions of cement and with the increase in density. Mortar being more compact does not easily absorb water and thus the trouble arising from the exposure of concrete to water pressure was avoided in the locks on the Illinois and Miss. canal by making the exposed faces of mortar, 1 part Port. cement to 2 parts sand for a thickness of 8″ and in the locks on the Coosa River by making the exposed faces of mortar, 1 part Port. cement to 3 parts sand, for a thickness of 4″. In the Cold Spring dam, as will be subsequently noted, the concrete next the molds was rammed with a flat, square pointed shovel thus forcing the stones back from the face and leaving practically a mortar surface.

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[1] Proc. Inst. C. E., Vol. CVII, p. 77.

Mortar faces, when employed, should be placed and rammed at the same time that the concrete is laid, for considerable difficulty is found in making cement mortar plasters stick to the concrete after the latter has set. Mr. Thomas T. Johnson states[1] that it is highly probable that the separation of the mortar plaster from the concrete at the surface of contact, is due to the different expansions of the two materials if indeed certain chemical actions do not effect the same result. Major W. L. Marshall, U. S. Corps Engs. states[2] in reference to plastering as follows: "In work under my direction as a rule no plastering is allowed, and where there must be a junction of divers mortars or concretes, where one element has already set, dovetails of sufficient size to take the stresses as tensile and compressive forces, relying not at all on cohesion, are prescribed. A single, double or triple wash of pure cement to fill up pores and make a more watertight structure is allowable, but plastering in thicker layers than a wash after the setting of the main mass is a poor resort in all cases, except where the forces acting are not great at any time, and are uniform at all times."

The shelling off of concrete, which occurs to a greater or less extent depending upon the climate and the compactness of the concrete, is due to the freezing of water in the pores and occurs also in all soft stone. The effect if circumstances seem to require it, can obviously be overcome entirely or to a great extent, by a cut stone, rubble, or mortar face for the exposed surfaces.

Owing to the lack of joints in which to distribute the expansion and contraction due to temperature changes, concrete is more liable to crack than brick or stone masonry. Major Marshall who was in charge of the construction of the concrete locks on the Illinois and Miss. canal states:[3] "All massive concrete work should be divided into sections by vertical planes at right angles to the longest dimensions or on approximately radical lines if curved, to determine in advance the planes of weakness along which cracks due to contraction in setting, or to changes in temperature, shall take place."

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[1] Journal Western Soc. C. E., Vol. I, p. 83.

[2] Journal Western Soc. C. E., Vol. I, p. 217.

[3] Journal Ass'n Eng. Soc.'s, Vol. XIII, p. 630.

The San Mateo dam was built in blocks of about 30′ x 30′ x 10′ which were so placed as to break joints both horizontally and vertically. A crack appeared in the Beetaloo dam[1] which was built without vertical joints, but the resident engineer gives as his opinion that it was not due to temperature changes but to settlement in the foundation. The cracks in the Maligakanda concrete reservoir[2] in Ceylon were due to changes in temperature, according to Sir John Fowler. These experiences and the cracking of the Cold Spring dam, as will be subsequently noted, tend to show that some provision for expansion and contraction is necessary.

As shown in the expert report of Messrs. Davis, Cross and Shunk on the plans for the Quaker Bridge dam, the curved form better accommodates itself to changes of volume due to changes of temperature.

A method that has been suggested to avoid damage from cracking is to make vertical joints in the concrete extending the whole width and depth of the dam, leaving a rectangular hole across the joint near the upstream face about one square foot in size, which is to be subsequently filled with asphalt. In an experiment[3] made by Mr. W. H. Stanger at the time when the methods for repairing the Maligakanda reservoir in Ceylon were being considered, a layer of asphalt ¾″ thick was spread so as to cover the faces of two concrete blocks. The blocks were afterward forced apart ⅜″ by wedges and the asphalt showed no signs of tearing after one month.

Fr. von Emperger gives[4] for concrete a change in length of 1 in 800 for a change in temperature of 180° F. This rate, however, varies with the density and the state of aggregation. He states that all masonry walls crack more or less, but in a dam or reservoir wall, the section should have such a great surplus of strength that cracks are not likely to occur and that in building concrete walls for any purpose, those which are slightly curved, or such as can be built in sections, are preferable.

It may be said that it is cheaper to build the structure entire and then if cracks appear, fill them up with grout under pressure, as was done with the cracks in the Beetaloo dam, or otherwise repair them. This method may be justified in small dams, but with a long structure the works may be seriously endangered and the process of repairing may need to be continued for some time, adding unforeseen expense to the work.

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[1] Proc. Inst. C. E., Vol. CXIII, p. 158.

[2] Proc. Inst. C. E., Vol. CXVI, p. 291. Eng. News, Vol. XXXII, p. 115.

[3] Proc. Inst. C. E., Vol. CXVI, p. 298. Eng. News, Vol. XXXII, p. 115.

[4] Eng. News, Vol. XXXII, p. 255.

Opinions differ as to the architectural beauty of a concrete dam, some claiming that it should be faced with rubble or cut stone when it is located where its appearance is an important item, others thinking that it can be made artistic by ornamentation, which is easily done by using molds of the required form.

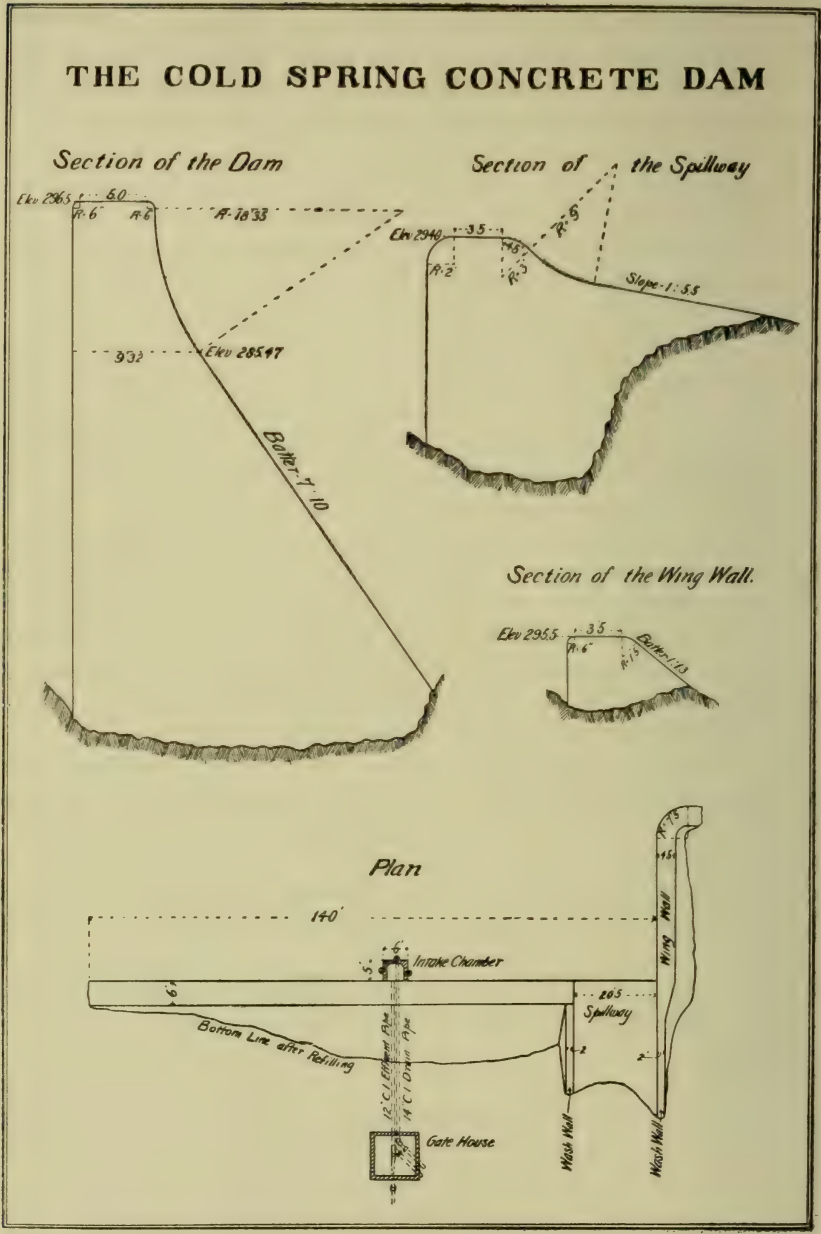
The Cold Spring dam spans a small creek with about 3½ square miles of drainage area forming a reservoir of 15,000,000 gallons capacity. The population of Cold Spring is about 3,000 and the village, which is 50 miles from New York City, lies on a sloping hill on the east bank of the Hudson River. The dam is 2½ miles back from the river and the spillway has an elevation of 294′ above it.

The dam is located between two ledge projections. The rock in these, as well as that underneath, was seamy, necessitating an average rock excavation of 10′. The dam, including the spillway but excluding the wing wall, is 140′ long on top, 85′ long at a distance of 20′ below the top, and 35′ long 40′ below the top. Its greatest height is 45′. The spillway is 20.5′ long and is situated upon the ledge on the south side of the creek in the same line with the dam.

The water, after leaving the spillway, is carried by wash walls and the natural rock on the ledge a distance of 100′ before it falls over into the old bed of the creek. The level of the spillway is 2.5′ below the top of the dam and, in order to prevent any possibility of the water overrunning the dam during freshets, the whole of the wing wall south of the spillway was left one foot below the top of the dam so that the water would run over this and waste on the ledge. The wing wall runs at right angles to the line of the dam and spillway and is 45′ long. Sections of the dam, spillway and wing wall and the plan are shown on page 18.

When the excavation was completed, the foundation was washed and scrubbed with brooms and any seams grouted with a mixture of 1 Portland cement to 2 sand. The whole bottom, before the laying of concrete, was covered with a thin layer of this mortar.

The broken stone for the concrete was crushed from the rock excavated at the site of the dam and from rock taken out of the ledge on the north side of the creek 80′ above the dam. The rock was a hard



THE COLD SPRING, N. Y., CONCRETE DAM.

gneiss which broke up well in the crusher. A sufficient amount of rock was blasted out before any concrete was laid so there would be no possibility of injury to the work from the blasting. There were three derricks, the first controlled the quarry above the dam, the second, the northern portion of the dam, and the third, the southern portion of the dam and the wing wall. The rock was delivered to the crusher platform by the first and second derricks and, after being crushed, fell on to the mixing platform. The concrete mixing was done by hand, the cement and sand being first mixed dry and then spread over the stone and the whole turned three times, water being added in the first turning. In the third turning, it was thrown into a scale box and then delivered on to the dam. It was placed in layers of 6″ to 8″ over the whole surface of the dam, the layers being made to slope upwards about 1 in 5 towards the downstream face. Each surface, if already set, was moistened and coated with a skim of mortar, 1 part Portland cement to 2 parts sand, before more concrete was added.

The proportions of the concrete were 1 Portland cement, 1½ sand, 3½ crushed stone for the first six feet at the bottom, then 1 : 2 : 4 for the next six feet, and 1 : 3 : 5 for the remainder of the dam. The outlet pipes, which consisted of one 12″ cast iron effluent pipe and one 14″ cast iron drain pipe, were surrounded by 3″ of mortar, 1 part Portland cement to 2 parts sand. The top of the dam was also covered for a depth of 3″ with this mortar.

The faces of the dam were made smooth by ramming just inside the forms with a flat square-pointed shovel, thus driving the stones back from the face and securing practically a mortar surface. Upon completion, the faces were also whitewashed over with a grout mixture of one cement to one sand, thus filling up any interstices that there might be. All corners were rounded off by a curve with a 6″ radius. It was found that these rounded corners required considerable extra labor and were difficult to make.

From a level of 30′ below the top of the dam, a saving in cement was effected by embedding boulders in the concrete, varying in size from ¼ to 1¼ cubic yards. These were placed far enough apart so that the concrete could be well rammed between them and were not put nearer either face than 2′. They were washed and covered with a skim of mortar, 1 part Portland cement to 2 parts sand, before the concrete was placed against them.

The forms or molds for placing the concrete consisted of 3″ × 8″ × 16′ upright pine timbers spaced 8′ apart, inside of which were placed 2″ × 8″ × 16′ tongue and proved pine boards planed on one side.

The uprights were held in place at the bottom by outside bracing, but above that were kept in position by ⅞″ bolts 18″ long placed 8″ in the concrete. These bolts had a nut at each end, thus when the outer nuts were unscrewed, the form could be removed and then by unscrewing the bolts from the inner nuts, they could be taken out, leaving these nuts in the concrete. The holes left by the bolts were rammed full of mortar.

The intake chamber from the foundation up to the pipes is built solid of concrete, above this it is made of brick. It is 5′ × 6′, outside measurements, with walls 12″ thick. The brick side walls of the chamber were bonded into the upstream face of the dam by leaving two vertical chases in the concrete, the width of the walls. The depth of these chases changed at every foot, being 6″ for the first foot, then 12″, then 18″, and then back to 6″ again, and so on up to the top. The intake chamber had three gate valves, one of 14″ on its upstream wall, its centre 31′ below the spillway, another of 16″ on its south side wall 20′ below the spillway, another of 16″ on its north side wall 4′ below the spillway.

The outlet pipes passed through a brick gate house 12′ × 12′ at a distance of 45′ below the upstream face of the dam. This contained the gate valves for regulating the amount of water supplied to the village or for draining the dam, and a 10″ Venturi meter so that a record of the amount of water furnished could be kept. The supply main running from this gate house to the village was a 10″ cast iron pipe.

The dam was designed and constructed by Mr. Francis Henry, the contractors were J. W. Hoffman & Co. The work of excavation was done in November and December, 1895, and the concrete was laid in the months of April, May and June, 1896. The total amount of rock excavation was 1732 cu. yds. and the total amount of concrete was 2306 cu. yds. The amount of cement used was 3245 barrels or an average for the whole dam of 1.4 barrels per cu. yd. of concrete. The use of mortar in covering the foundation, between the concrete layers and in the 3″ covering on top of the dam and the increased percentage of cement in the concrete mixtures used near the foundation, make this average higher than the normal for a 1:3:5 mixture although these items are offset to a certain extent by the saving in cement effected by the use of the embedded boulders. The contract price for rock excavation was $1.30 per cu. yd. and for concrete $3.10 per cu. yd., the village furnishing the cement. The cement cost $1.92 per barrel at the Cold Spring dock on the Hudson River. The wharfage, hauling to site of dam, tallying and the construction of a storage house at the dam were respectively 2, 15½, 3 and 4½ cents per barrel, which makes the cost of cement at the works $2.17 per barrel. The total average cost, therefore of the concrete per cu. yd. to the village was $6.14. ($2.17 x 1.4 added to $3.10.)

The dam was completed in June, 1896 and after one month was allowed to fill slowly. It has since been in constant use. In November, 1896, a crack appeared about 20′ from the northern extremity. The crack was about $\frac{1}{16}$″ wide and uniform. It extended in a nearly straight line across the top and ran down to bedrock in a nearly vertical plane on both of the upstream and downstream faces. It was repaired by drilling along the line of the crack on the upstream face one inch holes, 5 to 6″ deep and 3 to 3½″ apart vertically slanting downwards at an angle of 45° so that the bottom of one hole would be lower than the top of the adjacent hole below, and then ramming them full with brick clay and fine sifted sand. A chase was also cut along the line of the crack on top of the dam and filled with a grout of neat cement. No trouble from leakage has since been experienced, although the crack opens slightly in winter and closes in summer. It is believed that the clay has gradually silted up the crack.

The mortar covering on top of the dam has scaled off and separated from the concrete in some places, which tends to show as previously noted in the quotation from Major Marshall, that mortar coverings, when put on after the concrete has set, are not advisable unless bonded into the concrete.

DATA OF STREAM FLOW IN RELATION TO FORESTS.

GEORGE W. RAFTER, M.AM.SOC.C.E.

The development of water power in recent years has made everything relating to stream flow not only of interest and importance, but even of commercial value. It has, indeed, become necessary that engineers understand the laws governing stream flow—or more broadly, that division of the science of hydrology relating to stream flow—as a part of their professional stock in trade. It is proposed, therefore, to give in this paper some of the more useful physical data on which has been founded the view that with other conditions remaining the same, a forested area will yield a larger runoff in the issuing streams than will a similar area from which the forest covering has been removed. It appears the more desirable to present these data in a single compilation, because they are widely scattered—so far as the English literature of the subject is concerned never having been, with the exception of some of the annual reports of the Forestry Division of the U. S. Department of Agriculture and Bulletin No. 7 on Forest Influences of that Department, even approximately brought together. Abroad, on the other hand, the general question of forest influences has been extensively considered, not only as regards runoff of streams, but in its relations to many other questions as, for instance, the relation of the temperature under woods and outside ; the determination of rainfall under woods and outside ; the influence of the forest floor covering on the humidity of the soil and on evaporation from the same ; the amount of evaporation in the forest and without ; the quantity of ground water at different depths under woods and outside, with various aspects and under various kinds of trees ; the hygrometric condition of the air under woods and outside ; the rain and snow reaching the ground under woods and outside ; and manv other questions have been studied. In the United States, thus far, the foregoing divisions of the subject have, with one or two exceptions, been left untouched, although measurements of stream flow have, during the last few years, been carried on in many places.

Moreover, the subject has, for the reasons already assigned, attracted considerable attention, and been, in consequence, the object of a great deal of popular writing, in which the most diverse views have been expressed. Thus, it has been held that forests not only increase rainfall, but that they increase the total annual flow of streams, at the same time retarding the rate of surface runoff, thereby decreasing the severity of floods. The contrary view, that forests not only do not increase precipitation, but that they are without effect on rate of runoff, and hence that floods are quite as great from forested as from deforested areas, has also been strongly championed.

Henry Gannett's Paper on Forest Influences.—As one of the notable papers of the last ten or fifteen years, we may refer to a paper by Henry Gannett, formerly Geographer of the U. S. Geological Survey, which, possibly on account of the official position of its author, received an amount of attention from the public press, hardly justified by the contents of the paper. Mr. Gannett's paper probably attracted more popular attention than such publications ordinarily do, because of treating the conditions prevailing in our own country, the method being to select extensive areas widely separated geographically, and compare the amounts of precipitation during different periods. On this line, Mr. Gannett assumed as a prairie region the area comprising Iowa, northern Missouri, southern Minnesota, Illinois and part of Indiana, which he deemed to offer a proper field for comparison, because it is asserted that in the last thirty or forty years this area has been considerably reforested by natural growth. As a second area he selected the State of Ohio, originally heavily wooded, but which has been considerably deforested in the last sixty years. A third area comprised New England and parts of New York, which, originally densely wooded, then almost entirely deforested, is claimed by Mr. Gannett to now be largely grown up to forest again.

Mr. Gannett attempted to solve the question whether these assumed changes in forestration have been accompanied by changes in the quantity of rainfall.

It seems very obvious to the author that discussion on the lines here indicated must necessarily be barren of results. Sir Robert Rawlinson pointed out, many years ago, that as a broad proposition, climatic movements are on such an extended scale as to be generally beyond any influence which man can bring to bear. This proposition has never been successfully disputed. Indeed, the most skilled meteorologists of the present day assert that there have been no essential changes in climate anywhere, for the last two thousand years. This statement does not mean that swamp areas may not have been improved by cutting the timber and by drainage, but does mean that the general climate over areas as broad as those considered by Mr. Gannett, has remained, during the historical period, substantially unchanged. Again, it may be pointed out that ever since the invention of the rain gage by Leonardo de Vinci, rainfall records have been kept at a number of places abroad, some of them now extending over from one to two hundred years.[1] A study of such records shows that climate is, on the whole, persistently maintaining a certain average. It follows, therefore, that Mr. Gannett's method of treatment was fundamentally wrong.[2] There are a number of other papers of recent years, which, for various reasons, have been without effect other than to produce erroneous impressions in the popular mind.

The author has no desire to impeach the good intentions of any of the gentlemen presenting the views referred to. He recognizes that the chief difficulty with some of the recent writers on this subject—especially the authors of popular articles in the magazines—with which he is familiar, has been an evident lack of full acquaintance with the physical data really applying. One object of the present paper is, to some extent, to supply this lack by actually quoting these data, or, when too extended for citation, to indicate where some of the more important may be found.[3]

The author does, however, wish to point out as a tentative proposition, that the real reason why forested areas furnish more water in the streams for a given rainfall than do deforested and cultivated areas, is because forests, on the whole, consume less water than do deforested and cultivated areas. The data herein given are especially directed towards establishing this tentative proposition.

Reports of the Forestry Division, U. S. Department of Agriculture.—The work of this Department has been already mentioned, and it is referred to no further in this place than to recognize that some attempt to present adequate data has been made by the Division of Forestry, especially in the Annual Reports for 1888, 1889, etc., and in Bulletin No. 7, Forest Influences, already referred to. The Reports on Forestry, Volumes I and II, by Franklin B. Hough, may be also consulted, especially Volume I, for considerable data applying to the subject.

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[1] For convenient reference to a discussion of long rainfall records, see paper On Mean or Average Annual Rainfall and the Fluctuations to which it is Subject. By Alexander R. Binnie, M. Inst. C. E., Proc. Inst. C. E., Vol. CIX (1892), pp. 89–172.

[2] As to constancy of climate, see Russell's Meteorology, p. 101.

[3] It is not intended, however, to claim that all the data applying are cited herein. Merely such are given as have fallen under the author's observation.

DIVISIONS OF THE SUBJECT.

Following generally, though not exclusively, Durand-Claye in his *Hydraulique Agricole et Genie Rural*, we may discuss the subject under the following main divisions, namely: Rainfall, Temperature, Hygrometry, Evaporation, and Miscellaneous Data. This division is necessarily taken somewhat arbitrarily, one frequently running into another, and is made merely for the purpose of assisting somewhat in handling the data.

Before proceeding to the main discussion, the author may very properly indicate that while, as a tentative proposition, he has no doubt that, other conditions remaining the same, forest areas yield more water from a given rainfall than do similar deforested areas—and certain data derived from recent studies in the State of New York will be presented strongly substantiating this view—nevertheless it is recognized that some of the information really needed for final conclusions, is not yet gathered, and in consequence all final conclusions are necessarily delayed for many years—at any rate so far as the comparison of broad areas widely separated is concerned. Illustrations of why this is true will be given further on.

The foregoing definition of what seems to the author to be the present state of the data of stream flow in relation to forests, is made necessary because one of the mooted questions at the present time is as to the proper method of investigation to be pursued. The foreign studies of forest meteorology have been mostly carried on with reference to comparing temperature, hygrometric and other meteorological conditions in the forest, with outside conditions, the runoff of streams having been, generally speaking, only casually considered. On the contrary, Mr. C. C. Vermeule, of the New Jersey Geological Survey, asserts, in effect, that nothing satisfactory can be arrived at by the consideration of forest meteorology; he, indeed, ignores all of the accumulations of knowledge of the foreign meteorological stations and attempts to base conclusions entirely upon comparison of runoffs. So certain is Mr. Vermeule that this is the best method of procedure, that he rather makes a point of ignoring the data of forest meteorology.

The author's present view is that really satisfactory results will never be obtained until studies extending over a considerable number of years have been made, not only of the runoff of streams from similarly situated contiguous areas, the one forested and the other deforested, but of the meteorological conditions of such areas, including temperature, dew point, humidity, vapor pressure, precipitation, and wind velocity. By way of illustrating how these several elements vary in different parts of the United States, the following tabulations have been compiled from the Annual Reports of the Chief of the U. S. Weather Bureau for the years 1891–96, inclusive, the places therein given, at which complete observations have been kept, being selected with reference to showing what considerable variations in these elements prevail in different parts of the United States. It may be noted in passing that temperature, dew point, humidity, vapor pressure, precipitation, and velocity of wind, are the elements which determine the intensity of evaporation.

TABLE

SHOWING MEAN VARIATION OF YEARS 1891 TO 1896, INCLUSIVE, IN METEOROLOGICAL CONDITIONS IN DIFFERENT PARTS OF THE UNITED STATES.

NAME OF PLACE.	Mean Temperature.	Dew Point.		Relative Humidity.		Vapor Pressure.		Precipitation.	Wind—Average hourly velocity.
		8 A. M.	8 P. M.	8 A. M.	8 P. M.	8 A. M.	8 P. M.		
	F°	F°	F°	Per ct.	Per ct.	Inchs.	Inchs.	Inchs.	Miles.
Abilene, Texas	63.8	47.3	45.3	76.3	48.0	.374	.340	23.79	11.0
Albany, N. Y.	48.5	39.2	40.8	80.3	73.5	.292	.303	34.12	7.7
Block Island, R. I.	49.0	42.7	44.0	81.0	82.3	.320	.332	43.03	16.4
Boston, Mass.	49.5	39.1	40.8	73.3	72.7	.282	.299	39.32	12.3
Buffalo, N. Y.	47.0	38.1	38.7	74.7	70.5	.270	.276	37.25	11.9
Cleveland, Ohio	49.4	39.6	41.3	77.0	71.0	.290	.305	32.64	12.1
Denver, Col.	49.8	25.7	26.2	57.3	38.0	.160	.159	14.66	7.7
El Paso, Texas	63.4	30.8	24.7	49.3	23.8	.206	.166	7.11	9.7
Los Angeles, Cal.	61.9	46.0	50.8	80.0	64.5	.331	.383	14.23	3.6
Oswego, N. Y.	45.8	37.2	38.0	77.2	72.8	.266	.268	34.71	11.2

RAINFALL.

Meteorology may be defined as, broadly, the study of the atmosphere, its properties, motions and appearances, etc., an orderly arrangement of all the facts relating thereto constituting the science of meteorology.[1] The consideration of rainfall in relation to the runoff of streams may be considered a division of applied meteorology.

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[1] See Meteorology. By Thomas Russell, U. S. Engineer. Also, Meteorology; Practical and Applied. By J. William Moore.

Annual Rainfall.—In compiling rainfall records for purposes of comparison with runoffs of streams, it is the author's custom to arrange them in a water year beginning with the month of December, and ending with November. Such a year is again divided into three periods, December to May, inclusive, constituting the storage period; June to August, inclusive, the growing period; and September to November, inclusive, the replenishing period. During the storage period a large per cent. of the total rainfall appears as runoff in the streams, while in the growing period the percentage of rainfall appearing in the streams is very small, the bulk of the precipitation of that period being used up by plant life and evaporation from the surface of the ground. Usually the stored ground water is so drawn upon during this period as to produce a low water table. The replenishing period is an intermediate stage during which evaporation gradually decreases, and provided the rainfall is not deficient, ground water rises to its usual height. Naturally these periods run more or less into one another, depending upon whether seasons are advanced or retarded.[1]

The English writers on hydrology make a water year beginning with September and ending with August. This water year is again divided into a winter period extending from October to March, inclusive, and a summer period from April to September, inclusive, as best fitting the conditions of English climate. Mr. Beardmore in his Manual of Hydrology, proposes a water year of three periods of four months each, the first including November to February, inclusive, which he calls the winter division; the second, March to June, inclusive, the spring division; and July to October, inclusive, the summer division. This plan, Mr. Beardmore considers, gives better opportunity for comparison. As regards American climatic conditions, however, the author considers the division into storage, growing and replenishing periods, as given in the foregoing, on the whole, the best arrangement.[2]

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[1] For examples of rainfall and stream runoff records written up with reference to a water year extending from December to November inclusive, and divided into storage, growing and replenishing periods, see the author's several reports on Genesee and Hudson Rivers Storage Projects, in the Annual Reports of the State Engineer and Surveyor of New York for 1895–96, inclusive.

[2] See (1) Beardmore's Manual of Hydrology, pp. 2–81; (2) Harrison's paper On the Subterranean Water in the Chalk Formation of the Upper Thames, and its Relation to the Supply of London, in the Proc. Inst. C. E., vol. CV (1891), pp. 35–46; (3) Evans' paper On the Percolation of the Rainfall on Absorbent Soils, in Proc. Inst. C. E., Vol. XLV (1876), pp. 208–216; and (4) Graves' paper On Evaporation and Percolation, also in Proc. Inst. C. E., Vol. XLV (1876).

Rainfall data in the United States.—The chief sources of information as to rainfall in the United States are, (1) the Annual Reports of the U. S. Weather Bureau; (2) the monthly and annual publications of the various State Weather Bureaus; and (3) a large amount of manuscript records collected by the Smithsonian Institution previous to the establishment of the U. S. Weather Bureau in 1871, and now all in possession of the Chief of the Weather Bureau, and to be had by investigators on correspondence with his office. In addition to the stations of the U. S. Weather Bureau located at many of the important cities of the country, all of the States and Territories maintain Weather Bureaus and publish the results either in monthly or annual bulletins. The figures here given for the several States and territories, include also the U. S. Weather Bureau Stations in each division.

Alabama,	59	stations.
Alaska,	7	“
Arizona,	53	“
Arkansas,	51	“
California,	225	“
Colorado,	93	“
Connecticut,	21	“
Delaware,	6	“
District of Columbia,	4	“
Florida,	40	“
Georgia,	63	“
Idaho,	38	“
Illinois,	98	“
Indiana,	48	“
Indian Territory,	9	“
Iowa,	104	“
Kansas,	88	“
Kentucky,	50	“
Louisiana,	56	“
Maine,	19	“
Maryland,	44	“
Massachusetts,	80	“
Michigan,	73	“
Minnesota,	73	“
Mississippi,	53	“
Missouri,	109	“
Montana,	40	stations.
Nebraska,	133	“
Nevada,	55	“
New Hampshire,	26	“
New Jersey,	55	“
New Mexico,	40	“
New York,	97	“
North Carolina,	60	“
North Dakota,	39	“
Ohio,	143	“
Oklahoma,	22	“
Oregon,	69	“
Pennsylvania,	92	“
Rhode Island,	8	“
South Carolina,	45	“
South Dakota,	50	“
Tennesee,	47	“
Texas,	96	“
Utah,	43	“
Vermont,	16	“
Virginia,	45	“
Washington,	57	“
West Virginia,	45	“
Wisconsin,	66	“
Wyoming,	14	“

These figures give a total of 2,967 stations in the United States. In addition meteorological observations are made at a number of places not included in the foregoing list, such as agricultural experiment stations, colleges and high schools, etc., which easily make over 3,000 stations in all.

The State records are also tabulated in the Annual Report of the U. S. Weather Bureau. The Annual Report of the Chief of the Weather Bureau has, however, only been published in its present complete form since the year 1891, and it has the disadvantage to the student who desires up to date data, of being about two years behind in publication. If, therefore, one desires the recent data, it may be obtained from the Bulletins of the State and Territorial Bureaus, or by correspondence with the Chief of the U. S. Weather Bureau.

TEMPERATURE.

The foregoing statements as to sources of information about precipitation may be also taken as applying to temperature, hygrometry, etc., except that generally speaking the State and Territorial weather services have not gone extensively into hygrometric observations, the most of this work being confined to the stations of the U. S. Weather Bureau.

Mean Annual and Monthly Temperatures.—Mr. C. C. Vermeule in his several reports in the Annual Reports of the New Jersey Geological Survey[1] has stated that he has discovered a relation between mean temperature and the runoff of streams, and has published formulae based on the proposition that the runoffs of streams in the North Atlantic and Middle Atlantic States, vary with the temperature. In his opinion, mean annual temperature is the one controlling element determining ground evaporation.

A study of the table showing variation in meteorological conditions in different parts of the United States on page 26, may lead to the tentative conclusion, at any rate, that evaporation from the surface of the ground necessarily varies more broadly than Mr. Vermeule has apparently assumed. Thus, at Abilene, Texas, the mean temperature for the period considered in the table is 63.8°, while at El Paso, Texas, 450 miles west, the mean temperature is shown to be 63.4°. The mean dew point at Abilene is 47.3°, at 8 a. m., and 45.3° at 8 p. m.; while at El Paso it is 30.8° at 8 a. m. and 24.7° at 8 p. m. At Abilene, the relative humidities as per table, are 76.3 and 48.0 per cent.; while at El Paso they are 49.3 and 23.8 per cent. Vapor pressures at Abilene are 0.374 and 0.340; and at El Paso, 0.206 and 0.166; the mean rainfall at Abilene, 23.79 inches, and 7.11 inches at El Paso.

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[1] See Annual Reports of the State Geologist of New Jersey, (1890–96, inclusive), also, Report on Water Supply, Water Power, Flow of Streams and Attendant Phenomena, being Vol. III of the Final Report of the State Geologist of New Jersey.

The average hourly velocity of the wind at Abilene is 11.0 miles, and at El Paso, 9.7 miles. The country about both places is an elevated, dry region, entirely destitute of any water on the surface, and with very little timber, except small fringes along the margins of streams. It is perfectly clear to the author, from former experience in central and west Texas, that while the mean temperatures at these two places are substantially the same, the natural evaporation, except accidentally, is much greater at El Paso than at Abilene, although it may be pointed out that since wind movement is one of the controlling factors in evaporation, the lower average velocity of wind at El Paso will tend to reduce the average evaporation there. The difference in rainfall at the two places must also be taken into account in considering actual evaporation results. The broad proposition is that with equal rainfalls at the two places, other conditions remaining as shown by the tabulation, the evaporation either from water or from land surfaces will probably be greater at El Paso than at Abilene, although, possibly, the difference in wind velocity might accidentally make the two not very different. Similar comparisons can be made for other stations, as for instance between Block Island and Boston, where the mean temperatures are nearly the same, but where there are large variations in the other elements controlling evaporation. A coincidence in evaporation depths at these two places would merely mean an accidental balancing of all the forces influencing evaporation. We need, therefore, not only tabulations of mean monthly and yearly temperature as a necessary part of the study on this line, but tabulations of all the other elements influencing evaporation before we can decide.

Constancy of Temperature.—Temperature is a tolerably constant meteorological element, as may be inferred from the following statistics of mean annual temperatures.[1]

At St. Petersburg there is a record of temperature since 1743, the mean of which to 1875, inclusive, is 38.6°, the highest annual temperature being 42.4° in 1822, and the lowest 34.1° in 1815. At Paris, records kept from 1735–1890, inclusive, show a mean temperature for the whole period of 51.4°, the highest annual temperature of 57.6° occurring in 1781.

The following are the annual temperatures at Philadelphia for various groups of years from 1758–1889, inclusive:

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[1] For these long temperature records, see Russell's Meteorology, pp. 103, 104.

1758 to 1777	52.6°
1798 to 1804	54.2°
1829 to 1838	51.5°
1825 to 1845	53.1°
1846 to 1867	54.0°
1871 to 1889	53.1°

The lowest annual temperature at Philadelphia, 48.2°, was in 1836.

A study of temperature records, so far as the author has carried such a study, seems to show that there is no direct relation between the runoff of streams and mean annual temperature, although on this point he does not wish to be understood as yet passing a final judgment.[7]

HYGROMETRY.

Hygrometry may be defined as that branch of meteorology which is concerned with the determination of the amount of water present in the air in the form of vapor. The degree of humidity is its hygrometric state. When expressed as a per cent., it is relative humidity, but when the tension or elastic force of aqueous vapor which represents the pressure of all the vapor in the air above the point of observation is expressed in terms of inches of a mercury column, it represents the absolute humidity of the air.[2]

Variations in the Hygrometric State.—The hygrometric state varies greatly at different times of day and different seasons. For Diagrams and Tables illustrating these variations, see Durand-Claye's *Hydraulique Agricole et Genie Rural*, Chapter 7.

EVAPORATION.

General Formula for Evaporation.—So far as the English literature of the subject is concerned, the most satisfactory discussion of evaporation is that of Mr. Fitz Gerald,[3] whose formula for evaporation is as follows:

$$E = \frac{\left(V - v\right)\left(1 + \frac{W}{2}\right)}{60}$$

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[1] For a large number of temperature records in the State of New York, tabulated with reference to a water year from December to November, inclusive, and divided into storage, growing and replenishing periods, the same as for rainfall records; see the author's Reports on the Genesee and Hudson Rivers Storage Surveys, in the Annual reports of the State Engineer and Surveyor, as previously cited.

[2] Refer to Moore's Meteorology, Chapters XVI to XX.

[3] See paper On Evaporation. By Desmond Fitz Gerald. Trans. Am. Soc. C. E., Vol. XV (Sept. 1886), pp. 581–646.

In this formula, V means the maximum force of vapor in inches of mercury corresponding to the temperature of the water; v, the force of vapor present in the air; W, the velocity of the wind in miles per hour, and E, the evaporation in inches of depth per hour. It can be shown that there is going on nearly always a condensation of moisture from the air, upon any water surface. At the same time there is going on a loss of moisture from the water to the air. The intensity of both these operations depends upon the difference in temperature between air and water. When the temperatures of air and water in contact are the same, both processes stop. Evaporation is, therefore, the measure of the difference of these two exchanges. The experiments of Mr. Fitz Gerald and others have shown that evaporation from water surfaces is subject to a definite law, expressed by the formula just given, but evaporation from the ground has never been reduced to any such simple expression. Various difficulties arise which apparently render it impossible to make a single expression covering all the phenomena involved. If the surface of the ground be kept constantly wet, evaporation therefrom goes on substantially as from water surfaces. The main difficulty, therefore, in reducing evaporation from the ground to a simple formula, is largely due to uncertainty of the water supply. The demands of evaporation from the surface of the ground are continuous, the same as from other surfaces, but constant interruptions by either complete or partial exhaustion of the available supply, complicate the action so much as to render expression by formula apparently impossible. Varying demands of vegetation at different seasons also further complicate the problem.

Evaporation Data.—Evaporation data applying to water surfaces at Boston may be found in Mr. Fitz Gerald's paper already referred to. For evaporation in the basin of the Croton river, refer to the author's paper On the Water Resources of the State of New York, No. 24 of the Water Supply and Irrigation papers of the U. S. Geological Survey; and for evaporation data at Rochester, refer to the Annual Reports of the Executive Board of the City of Rochester, 1891–98, inclusive; also to the author's Report on the Genesee River Storage Surveys, and to the paper on the Water Resources of the State of New York.

For evaporation data abroad, at a number of foreign points, refer to Beardmore's Manual of Hydrology and to Durand-Claye's *Hydraulique Agricole et Genie Rural*, page 257, where evaporation data applying to Paris, Turin and Lake Fucino, may be found. Tables showing the hourly variation of evaporation may be found in Mr. Fitz Gerald's paper, and in Durand-Claye. A number of tables of evaporation at foreign points are given in the author's Report on the Upper Hudson Storage Surveys for 1896.

Condition Chiefly Affecting Evaporation. — According to Durand-Claye, the evaporation from a water surface depends upon several elements outside of temperature, as for instance, the hygrometric state exercises a certain action, but the agitation and constant change caused by wind movement exercises the greatest influence of all, because of quickly removing layers of air in contact with the liquid surface, substituting others in place of them.[1]

Double Significance of the Term Evaporation.—Mr. Beardmore points out in his Manual of Hydrology that the term evaporation is, in reality, of double significance, the engineer regarding it as much with reference to the quantity lost when the soil is wetted by rain or dew, as the amount merely evaporated from exposed surfaces of water, which is the evaporation of the meteorologist. Writers on general meteorology having thus far only casually taken into account land evaporation.

Negative Evaporation.—This term may be taken to mean that when the temperature of the dew point is higher than that of the evaporating surface, water is deposited on that surface. Studies of runoff data in comparison with rainfall and without regard to forest meteorology indicate that on some watersheds this is a frequent condition. We need forest meteorology therefore to correct the uncertainty of the runoff observations in this particular.

Evaporation from Naked Soil.—As already pointed out, evaporation from the ground follows substantially the same law as evaporation from water surfaces, varying in accordance with temperature, hygrometric state, wind movement, etc., and further varying with the nature of the soil. Experiments on this line were made by Maurice at Geneva, Switzerland, in 1796, and by De Gasparin, at Orange, France, in 1821, from which were determined the ratios between evaporation from the soil and rainfall for those years, as 0.61 for the first, and 0.88 for the second. More recent studies show that these old determinations have historical value only. Very recent French experiments are those of Marie-Davy, made at the Municipal Observatory of Montsouris, as recorded in the Annual Report of the Montsouris Observatory. Probably the best experiments are those made in England, where the percolation through drain gages has been observed at the

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[1] Also refer to Fitz Gerald's paper, Beardmore's Hydrology, etc., for more extended statements as to conditions affecting evaporation.

Rothamsted Agricultural Experiment Station and at other places for a number of years.[1] We may cite some of the results obtained by Mr. Graves, and detailed in his paper On Evaporation and on Percolation before the Institution of Civil Engineers, Vol. XLV, as already cited. The following summary of results for fourteen years, all in inches, includes the rainfall and percolation through turfed soil, and through a bed of deep, fine sand, the soil and sand being contained in a strong, open topped, water tight, slate box or tank, with an area of one square yard and thirty-six inches in depth. The soil is turfed over and the grass occasionally cut.

		PERCOLATION.		EVAPORATION.		
Year.	Rainfall.	Ground.	Sand.	Ground.	Sand.	Water.
1860	32.56	10.76	24.36	21.80	9.10	21.06
1861	23.63	5.71	16.36	17.92	7.27	25.01
1862	26.58	8.55	21.18	18.03	5.40	17.33
1863	19.77	3.76	16.41	16.01	3.36	18.27
1864	15.89	3.82	12.64	12.07	3.25	18.64
1865	29.25	11.15	27.82	18.10	1.43	20.12
1866	31.70	12.59	28.11	19.11	3.59	18.82
1867	27.44	5.16	22.42	22.28	5.02	20.06
1868	23.31	7.11	20.20	16.20	3.11	26.93
1869	24.56	8.05	22.14	16.51	2.42	19.06
1870	20.40	7.23	18.70	13.17	1.70	20.40
1871	24.08	6.19	20.08	17.89	4.00	19.58
1872	37.17	12.03	30.05	25.14	7.12	22.92
1873	23.77	4.05	20.12	19.72	3.65	20.40
Mean	25.72	7.58	21.41	18.14	4.31	20.61

Experiments at Geneva, New York.—Experiments at the N. Y. State Agricultural Experiment Station at Geneva may also be referred to. In August, 1882, the Geneva Agricultural Experiment Station began a set of observations on the amount of percolation through three drain gages, gage No. 1 being covered with sod, with the grass kept short by frequent cuttings; gage No. 2 was kept free from all vegetation, its surface being left undisturbed, while the surface of No. 3 was kept in a loose and fine condition by frequent stirrings with a trowel. These experiments were continued from 1882 until 1890.[2]

The following table gives the average rainfall and percolation through each of the three gages for the period from August 1, 1882, to November 1, 1887, inclusive:

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[1] Extended references to these data may be found in the author's Report on the Upper Hudson Storage Surveys for 1896.

[2] For full detail see the Annual Reports of the New York Agricultural Experiment Station at Geneva from the Second (1883) to the Ninth (1890).

	Rainfall. Inches.	DRAINAGE. Sod. Inches.	Bare Soil. Inches.	Cultivated Soil. Inches.	Average. Inches.
January	0.939	0.289	0.374	0.351	0.338
February	1.596	1.021	0.831	0.610	0.821
March	1.030	0.669	0.687	0.636	0.664
April	1.834	0.655	0.968	0.964	0.862
May	2.180	0.248	0.598	0.929	0.592
June	2.710	0.244	0.287	0.738	0.423
July	4.146	0.001	0.742	0.868	0.537
August	3.032	0.002	0.629	0.878	0.503
September	1.952	0.027	0.493	0.762	0.427
October	1.733	0.016	0.537	0.611	0.388
November	1.722	0.192	0.360	0.846	0.466
December	0.850	0.098	0.442	0.372	0.304
	23.724	3.462	6.948	8.565	6.325

A new set of drain gages, provided with an artificial water table, were set up at Geneva in 1888, but thus far the results gained with them have not been discussed at length in the Annual Reports. As to whether or not the Geneva gages are still in service is unknown, although the author cannot but think that some agricultural station, at any rate, should carry out to final conclusions such a set of experiments as was begun at Geneva.

Risler's Paper on Evaporation from the Soil.—Probably the most satisfactory data as to evaporation from soil are those contained in a paper *Sur l'Evaporation du Sol*, by E. Risler, published in the *Bibliotheque Universelle et Revue Suisse*, *Archives des Sciences Physiques et Naturelles*, for September, 1869. In this paper Risler gives the detail of experiments carried out by him on his estate at Caleves near Nyon, Switzerland. The following matter is translated from the paper :

"Meteorologists have made many attempts to procure the proportion of the rainfall which is returned to the atmosphere by evaporation, either directly by evaporation from the ground, or by passing through vegetation. To determine these facts, different methods have been used.

"The method which I have employed for solving the problem, differs completely from those used by other experiments. Thus, my method consists in gaging the amount of water issuing from under-drains, draining a certain known area, and in comparing the amount of drainage water with the amount of rainfall on the same surface, the difference between the two giving the quantity of water evaporated.

"In order to successfully apply this method, two principal conditions must be observed :

"(1) No other water should be allowed to come in contact with the earth except the rainfall directly upon the surface. For my experiments, a piece of land 12,300 square meters, situated upon the highest part of a clay plateau, was selected. No water could reach this area from the neighboring soil.

"(2) It is necessary that all the water which is not evaporated be received by the drains, this second condition being equally important with the first. In the area experimented upon, the sub-soil is so compact that the drains which are laid at a depth of 47 inches and 33 feet apart, must necessarily receive all the water which penetrates beneath the surface, and which is not evaporated.

"The experimental field fulfills all the necessary conditions. It has no trees. In 1867 two-fifths of the area was planted to potatoes, two-fifths to wheat, three-twentieths to Lucerne-grass, and one-twentieth in ditches carrying the drainage water. In 1868, two-fifths were in wheat, two-fifths in clover, three-twentieths in Lucerne-grass, and one-twentieth in ditches. Gagings of the drainage water were made at noon each day at a point below where all the drains come together.

"The following table gives the quantity of rainfall for each month of the years 1867, 1868, the runoff of the drains and the quantity of water evaporated. These figures show that in 1867 the evaporation was 68.75 per cent. and in 1868 73.17 per cent. of the rainfall, or making certain corrections because of the water in the soil. we have the final figures for 1867, 70.75 per cent., and for 1868, 70.17 per cent."

RISLER'S EXPERIMENTS ON THE YIELD OF WATER FROM UNDERDRAINED AREAS.

MONTH.	1867.			1868.		
	Rainfall in Inches.	Runoff by the drains.	Evaporation.	Rainfall in Inches.	Runoff by the drains.	Evaporation.
January	5.413	4.048	1.365	2.391	0.896	1.494
February	2.486	1.679	0.807	0.374	0.337	0.037
March	8.139	3.716	4.423	3.697	1.920	1.776
April	6.172	2.804	3.367	2.598	0.159	2.439
May	3.972	0.709	3.263	1.649	0.045	1.604
June	3.179	0.028	3.151	1.862	0.000	1.862
July	1.238	0.000	1.238	4.704	0.000	4.705
August	1.958	0.000	1.958	2.906	0.000	2.906
September	3.904	0.000	3.903	6.216	0.081	6.135
October	3.693	0.130	3.562	4.194	0.958	3.236
November	0.293	0.000	0.293	1.978	0.784	1.229
December	1.545	0.000	1.545	8.051	5.727	2.324
Total for year	41.992	13.114	28.875	40.620	10.907	29.747

As shown by the table, the evaporation loss in 1867 was 28.875 inches, and in 1868, 29.747 inches. These figures are especially interesting as representing the total evaporation loss from a catchment basin with an annual rainfall of from 40 to 42 inches. They may be instructively compared with the evaporation data of Genesee river given further on.

Combined Evaporation of Soil and Growing Plants.—The foregoing extracts from Risler's paper *Sur l'Evaporation du Sol* and the accompanying table, serve to illustrate the combined evaporative effect of soil and growing plants. By way of illustrating why plants contribute powerfully to increase evaporation, we may refer to the following data from Durand-Claye giving the relation between the evaporating surface of a number of agricultural crops and the unit area of the soil on which they grow:

Potatoes,	6.88	Wheat	10.95
Lucerne,	7.02 to 12.40	Grass,	12.40
Rye,	6.50 to 8.24	Clover,	16.36
Indian corn,	8.00 to 22.40	Grape vines,	4.94
Branching cabbage,	8.00	Oak trees,	9.00
Oats,	9.11	Fir trees,	11.75

Consumption of Water by Growing Agricultural Crops.—In a paper, *Recherches sur l'Evaporation du Sol et des Plantes*, Risler has given the results of further experiments at his estate in Switzerland, extending considerably the data of the paper already cited, his experiments being carried out specially with reference to ascertaining the mean daily consumption of water by growing agricultural plants, as well as by vineyards and two kinds of forests.[1]

The following matter relating to Risler's experiments is condensed from Ronna's *Les Irrigations*:

"By way of confirming the results of investigations as to the water consumed by growing plants, etc., carried out at the Agricultural Experiment Station of Rothamsted, England, Risler has shown the different methods employed by him in 1867 and 1868. By a continuation of these experiments in 1869–72, he has shown the mean daily consumption of water by lucerne, wheat, oats, clover, meadow grass, etc. One of his interesting conclusions is that winter wheat would have consumed daily, from April to July, 1869, 0.10 of an inch of water per day for 101 days, or over 10 inches for the growing season. The experiments on water content of soil show that for the year 1869, the crops must have taken a small amount of water from the ground which, with the rainfall, was sufficient to produce a satisfactory crop for the meteorological conditions prevailing that year. . . . .

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[1] Risler's experiments as detailed in the two papers cited may be taken as classical. In the author's opinion they are the most thorough determinations thus far made. They are quoted with approbation by Durand-Claye in his *Hydraulics Agricole et Genie Rural*, and by Ronna in his recent work, *Les Irrigations*, and by other foreign writers. They have also been quoted by various American writers, but so far as known to the author, the original papers have not been much studied in this country.

For oats there was needed in 1870, according to Risler, a quantity of water 250 times the weight of dry material contained in the crop. In 1871, clover transpired 263 units of water to produce one unit of dry substance, and English ray-grass 545 units of water for one unit of hay containing 15 per cent. of water. For this last the quantity of water corresponds to 0.276 inches in depth per day.

"Risler observed, furthermore, that following rains or wettings, transpiration of plants increases, gradually diminishing in proportion as dryness increases, other conditions remaining equal. When the water given off by the leaves is less than that taken up by the roots, growth is active, while under the contrary condition, plants wither.

. . . .

"In a general way, the consumption of water by plants is more regular in clay soils than in sandy. Hellriegel states that in a sandy soil plants begin to suffer from drouth when the soil does not contain more than 2.5 per cent. moisture. Risler finds that the approximate limit for clay soils is 10 per cent., although in clay soil part of the water escapes absorption by the roots.

"Taking as a basis the observations made on the crops raised at Caleves, Risler expresses the mean daily consumption of water as a depth on the cropped area, as follows:

Meadow grass	requires	from	----0.134 to 0.267	inches.
Oats ---------	"	"	----0.140 to 0.193	"
Indian corn --	"	"	----0.110 to 0.157	"
Clover -------	"	"	----0.140 to	"
Wheat -------	"	"	----0.106 to 0.110	"
Rye----------	"	"	----0.091 to	"
Potatoes -----	"	"	----0.038 to 0.055	"
Vineyards----	"	"	----0.035 to 0.031	"
Oak trees ----	"	"	----0.038 to 0.035	"
Fir trees -----	"	"	----0.020 to 0.043	"

"Risler determined the consumption of water on a meadow of one hectare (2.47 acres) of very thickly turfed English ray-grass as

281 millimeters (11.06 inches), amounting to a daily depth of 0.267 inches. This consumption applies to a meadow well provided with water during the warmest season of the year. The experiments showed that on cloudy days evaporation was reduced to about one-fourth of the mean, that is, to 0.069 inches per day.

"In Switzerland the fields begin to grow green the latter part of March, and the hay harvest occurs in June; hence the growth of the plant takes place in April and May. The amount of the hay crop, the rainfall and mean temperature during these months in the six years from 1866–72, as observed by Risler, are given in the accompany-table, which is generally self-exlpanatory. The point is brought out very forcibly by Risler's experiments, that hay crops depend more on the quantity of rain than on temperature; thus, in 1867, when the temperature of the two months was the lowest, but the rainfall high, the meadows yielded abundantly, while in 1868, with a high temperature, and medium rainfall, the crop was satisfactory because the soil had water in reserve, the drains continuing to flow until the end of May that year."

RISLER'S EXPERIMENTS ON THE YIELD OF MEADOWS.

YEAR.	Yield of hay per acre in pounds.	Rainfall in Inches.		Dates when Drains ceased to deliver Water.	Temperature C°.		
		In April and May.	Mean per day.		In April.	In May.	For the two Months.
1866	3864	9.533	0.156	End of May.	12.42°	13.70°	13.06°
1867	3375	10.078	0.165	" "	10.67	13.11	11.89
1868	2842	4.248	0.070	" "	9.00	18.72	13.86
1869	2753	6.303	0,103	End of March.	10.80	15.67	13.23
1870	1288	1.271	0.021	" "	9.80	16.12	12.96
1871	2975	3.864	0.063	End of April.	11.20	13.91	12.46
Means	2849	5.883	0.096		10.62	15.20	12.91

Transpiration of Plants.—This division of the subject, while discussed extensively by the great botanical writers Von Sachs, Kerner and Oliver, and others, has not as yet been reduced by the botanists to numerical relations. The best general discussion of the how and why plants transpire water, may be found in Kerner and Oliver's Natural History of Plants, where every phase of the question has been touched upon. Reference may also be made to Von Sachs' Physiology of Plants, and Bessey's Botany, where various facts relating to the general subject and statements of experiments are given. Bulletin No. 7, Forestry Division of the U. S. Department of Agriculture—Forest Influences—may also be consulted. Among other interesting data there given, we may refer to Wollny's experiments on water transpiration of agricultural crops, which are of special interest, not only because they are the most recent determinations, but because of verifying the earlier work of Risler. They are as follows :

Crop.	Growing Period.	Total Water inches.	Daily Consumption of Water, inches.
Winter rye,	April 20—Aug. 3, 1879,	11.44	0.109
Summer rye,	April 20—Aug. 14, 1880,	13.25	0.114
Barley,	April 20—Aug. 3, 1879,	12.00	0.114
Peas,	" "	13.88	0.132
Oats,	April 20—Sept. 14, 1880,	15.12	0.103
Beans,	April 10—Sept. 10, 1880,	13.80	0.096
Red clover (1st season)	April 20—Oct. 1, 1879,	13.56	0.083
Red clover (2d season)	April 20—Oct. 1, 1880,	17.95	0.109

The following figures of water consumption of certain plants for the growing season, computed from the data of Hales, Schleiden, Schübler and Höhnel and others are also mostly derived from Bulletin No. 7 on Forest Influences, certain apparently incongruous data there given being omitted :

Plant.	Total use of water inches.	Water per day inches.
Sunflower,	15.2	0.130
Cabbage,	14.4	0.120
Vineyard,	4.0	0.030
Hop vine,	5.6	0.050
Clover and oats (mixed)	12.7	0.098
Grass,	14.0	0.080
Beech trees,	9.2	0.050
Mixed forest,	3.8	0.021
White poplar,	7.7	0.051
Long grass,	40.7	0.166
Short grass,	27.4	0.112

The figures for long grass and short are based upon the determinations made at Emdrup, Denmark, as cited by Beardmore, and include the months from March to October, inclusive.

Elaborate investigations on transpiration of forest trees were made by F. B. Höhnel at the Austrian Experiment Stations in 1878, an abstract of which may be found in Bulletin No. 7, Forest Influences. From the figures there given, it appears that forest transpiration from day to day has a wide range. Thus, a birch standing in the open, and having 200,000 leaves, was calculated to have transpired on hot summer days from 700 to 900 pounds of water, while on other days not more than 18 to 20 pounds. A 35 year old beech tree with 3,000 leaves, was computed to transpire about 2.5 pounds per day from June to November. Assuming that 1,600 such trees might be found on an acre, the total transpiration might amount to about 600,000 pounds per acre, for each growing season of 150 days. A 50 to 60 year old beech, with 35,000 leaves, transpired about 22 pounds daily, and with 500 such trees on an acre, the transpiration for 150 days would amount to 1,650,000 pounds. This latter figure is equivalent to 7.28 inches on ground area.

The following table is derived from Höhnel's data, given at page 79 of Bulletin No. 7, Forest Influences:

Kind of tree.	Relative transpiration.	Transpiration for growing season. Inches on ground area.
Ash	100.00	9.18
Birch	90.2	8.18
Beech	89.7	8.13
Hornbeam (Ironwood)	86.0	7.75
Elm	80.8	7.32
Maple	69.1	6.24
Norway maple	60.1	5.45
English oak	67.9	6.15
Oak	48.3	4.37
Aspen	94.2	8.54
Alder	91.6	8.30
Linden	86.8	7.87
Larch[1]	123.3	11.17
Norway spruce	13.8	1.25
Scotch pine	11.9	1.08
Fir (balsam)	9.2	0.83
Austrian pine	6.9	0.63

The foregoing figures give an average transpiration for the deciduous trees mentioned of about 7.3 inches, while for the four conifers the average may be taken at about 1 inch.[2]

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[2] This is probably the European larch. The American larch (tamarack) grows naturally in swamps and is like the swamp ash, a large consumer of water. Tamarack is however a slow grower, and probably 11.17 inches per year is a large figure even for a swamp habitat.

[2] In the original table from which these data are derived, the results are in kilogrammes evaporated per 100 grammes of dry leaves. The data stated in this form not furnishing any basis for practical computation, in order to reduce them to inches on the ground area, the percents have been computed by assuming ash at 100, and the transpiration of the other species as proportionate to the number of kilogrammes per 100 grammes of dry leaves, in this way obtaining the second col-

As shown by all the recent experiments, evaporation from the surface of plant foliage is much less rapid per unit area than from water surfaces. An extended discussion of why this is so is given by Kerner and Oliver in their Natural History of Plants. As shown, however, by the data on page — the area of foliage is much greater than the ground area on which the plant stands, in the case of Indian corn the increase in area amounting, as a maximum, to 22.4 times the ground area. It is because of this immense transpiring or evaporating surface that some plants throw off more water than can be evaporated from the ground area on which they stand.

Experiments of Baldwin Latham.—Baldwin Latham, the great English Sanitary Engineer, has stated[1] that beginning in June, 1870, he discharged sewage upon a definite area, planted with ray-grass, at the Beddington-Croydon sewage farm, the grass being so placed in a water-tight tank six inches deep, and provided with suitable underdrains, with means of collecting the runoff of the drains, that definite results could be obtained. From June 18, 1870 to June 12, 1871, a period of 360 days, during which time 20.03 inches of rain fell, the water evaporated from a square yard of surface amounted to a depth of 186.3 inches over the area in a year. Experiments on another plat during the year 1871–72, gave for a period of 370 days, with a rainfall of 24.98 inches, a total application of water amounting to a depth of 91.2 inches over the area. These experiments indicate what large

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umn of the table. Inches on the watershed have been computed from the data as to beech trees, assuming the transpiration, for the growing season, of the average beech tree at 8.13 inches on the ground area, the other transpiration depths being made proportionate. The results in this form are, of course, only approximate, but are interesting as verifying previous data. In any case, it should be borne in mind that with the infinite variations of climate and soil, such data are, and must ever remain, more or less approximative. These data are also given by Mr. Nisbett in his recent work, Studies in Forestry, but with figures differing somewhat from those in Bulletin No. 7. In the absence of Höhnel's original paper the author has no means of determining which set is right. In any case these figures are merely given for illustrative purposes, and slight variations do not therefore especially affect the final results.

Moreover it may be pointed out that "inches on the ground area" of this table are computed on the supposition that all forests have the same density as the assumed typical beech forest. As a matter of fact the density of different forests will vary greatly, the average being less than the beech. On this basis the figures for inches on the ground area are undoubtedly somewhat in excess of the truth.

[1] In discussion of Mr. O'Meara's paper "On the Introduction of Irrigation in New Countries, as Illustrated by Northern Colorado." Proc. Inst. C. E., Vol. LXXIII (1893).

quantities of water may be evaporated by grass crops, provided a full supply is furnished.

Evaporation from Long and Short Grass.—Observations as to the evaporation from grass crops have been made at Emdrup, Denmark.[1] According to these observations the mean evaporation from a water surface for a period of 11 years was 27.9 inches. The mean evaporation from short grass for a period of 8 years was 30.1 inches, while for long grass for a period of 8 years the mean evaporation is given at 44.0 inches. Inasmuch as these interesting tabulations may be readily referred to in Beardmore's Hydrology, they are not reproduced here.

MISCELLANEOUS DATA.

In the foregoing chapters we have indicated some of the more important divisions of the subject, with brief citations of illustrative material. In the present chapter certain further data of value will be introduced.

According to determinations made by N. B. King at the Wisconsin Agricultural Experiment Station, barley requires from 12.19 inches of water to 23.52 inches; oats from 19.00 to 19.69 inches; corn from 25.00 to 26.39 inches; clover, 29.73 inches; and pease 26.89 inches.[2]

Experiments on the same general lines as Risley's have also been made at the Iowa Agricultural College.[3]

Number of Trees on an Acre.—According to Höhnel, in a forest of 50 to 60 year old beeches there may be as many as 500 on an acre, or there would be a tree to every 87 square feet, each tree occupying, then, a square of 9.4 feet per side. For a 35 year old beech forest, Höhnel states the possible number at 1,600 per acre, which would allow 27 square feet per tree, or each tree occupies a square with side length of 5.2 feet. By way of showing the number of large trees on an average acre of Adirondack forest, we may cite the following table from Mr. Pinchot's The Adirondack Spruce. The table gives the average number of trees per acre, percentage of each species, and average diameter of all sound trees over 10 inches in diameter, breast high, as determined from extended measurements averaged over 1,046 acres:

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[1] See Beardmore's Manual of Hydrology, p. 296.

[2] Refer to Ninth An. Rep't Wis. Ag. Experiment Sta., p. 96.

[3] Refer to Bulletin No. 32 of the Iowa Agricultural College Experiment Station, 1896.

Name of Species.	Average number of trees per acre.	Per cent of each species.	Average diam. breast high inches.
Spruce,	31.40	42.77	13.0
Birch,	14.00	19.06	17.1
Beech,	10.00	13.62	13.2
Hard maple,	6.10	8.30	13.9
Hemlock,	4.60	6.26	16.7
Balsam,	4.20	5.72	11.4
Soft maple,	2.60	3.54	13.6
White pine,	0.18	0.24	18.4
Ash,	0.16	0.22	12.9
Cedar,	0.12	0.16	14.5
Cherry,	0.08	0.11	15.3
Average and total,	73.44	100.00	14.5

In the following table, also, from Mr. Pinchot's book, we have a statement of the average number of trees of small diameter, other than spruce, as derived from measurements of 12 square acres :

	DIAMETER BREAST-HIGH IN INCHES.									
	2	3	4	5	6	7	8	9	10	Total.
	AVERAGE NUMBER OF TREES PER ACRE.									
Birch,	5.3	5.2	4.8	2.8	3.1	1.9	1.6	1.8	1.3	27.6
Beech	11.2	11.0	6.8	4.8	5.4	4.1	3.4	3.3	2.7	52.6
Hard Maple,	1.1	1.9	1.3	2.0	2.4	2.5	2.3	2.0	2.0	17.6
Hemlock,	0.4	1.4	1.4	1.8	1.7	0.4	0.8	0.3	0.6	8.7
Balsam,	11.6	14.2	12.3	8.1	6.5	4.3	3.3	1.8	1.6	63.6
Soft Maple	0.5	0.4	0.5	0.3	0.3	0.3	0.1	0.6	0.6	3.4
Pine,			0.08	0.08						0.2
Ash,	0.08	0.17	0.9	0.5	0.8	1.7	0.9	0.5	0.5	6.1
Cherry,			0.08		0.08					0.2
Total,	30.1	34.3	28.1	20.2	20.2	15.3	12.4	10.2	9.2	180.0

By way of showing the reasonableness of the proposition that forests do really consume much less water than agricultural areas and that per consequence, the runoff from forested areas will be considerably larger than from farming areas with the same rainfall we may cite the following data from Mr. Pinchot's book, as to rate of growth of forest trees, from which it will be seen that the growth of trees in forests is very slow, the yearly addition of woody material being apparently much less than for well farmed crops in agriculture. Assuming the truth of this proposition, a very strong argument is at once obtained as to why forests need less water than farm areas.

The following table gives the rate of growth in diameter, of several species actually associated in the Adirondack forest, as determined by careful measurements in Ne-Ha-Sa-Ne Park, where all of Mr. Pinchot's studies have been made:

Species.	Number of trees measured.	Average diameter inches.	Number of years required to grow one inch in diameter.
Birch,	78	12.5	20
Beech,	16	12.7	13
Hard maple	16	15.0	16
Hemlock,	141	16.6	25
Balsam,	63	10.5	13
Soft maple,	21	10.5	17

Taking into account the foregoing figures of Mr. Pinchot, as to actual number of trees per acre of average Adirondack forest, it seems tolerably clear that Höhnel's estimates of from 500 to 1,600 small beeches is rather large for average Adirondack forests, although for the extremely heavy forests of Washington and Oregon, and some of the other forest producing areas of the United States, Höhnel's figures would probably be too small. As furnishing further data as to number of small trees, the following information given to the author by J. Y. McClintock, C.E., in a private letter, is pertinent. Mr. McClintock's measurements having been made during the last two years, in connection with his work for the New York State Forest Preserve Board. Mr. McClintock says:

"I have examined the several hundred counts made by me in the southerly half of the Adirondack forest, and covering, in a general way, more than one hundred thousand acres. Without averaging them all I find that 84 different acres scattered over the McComb Purchase, Moose River Tract, Vrooman Patent, Arthursboro and Remsenburg Tracts, etc., give an average of 383 small trees, from less than 10 inches down to about 1 inch. I have included in these counts anything which had any probability of ever growing into a tree. Twenty-six counts on the Arthursboro Tract averaged 446 small trees per acre, while Township No. 10, Moose River Tract, with several hundred counts in all parts, averages 437 small trees per acre. From this I judge that a fair general average of the North Woods is about 400. In addition to this growth, I wish to remind you of the prevalent underbrush, often dense, particularly of witch hopple, which over large areas frequently covers the whole ground with its broad leaves; also of the less important sarsaparilla, nettle, moonshine, tansy, fireweed, ground hemlock, smartweed, steeple bush, spignet, raspberry, blackberry, ferns, and others. Some of these do not thrive in the dense woods, but all assist in clothing the surface with vegetation. The mosses are a feature of the vegetable growth in the woods, and the thick grasses in the extensive beaver meadows."

Mr. McClintock's observations, as briefly detailed in the foregoing, are interesting, as coming from a man who recently tramped many hundred miles in the Adirondack forests of New York.

The Relation between Rainfall and Runoff.—In order to determine the relation between rainfall and runoff in cases where both rainfall and runoff data are available for a series of years, it is the author's custom to write the rainfall with the corresponding runoffs in a descending series, without reference to the sequence of the years, and plot therefrom a curve with rainfalls as abscissas, and runoffs as ordinates, thus constructing what may be termed the runoff curve for each stream. On drawing such a curve, we find that as in other similar cases, the runoff for the given rainfalls group themselves above and below the curve, the extent of the departures from a true curve depending upon the accuracy of the data. If there are wide variations, we may conclude that there are large errors, either in the rainfalls or runoffs, or both. If, on the contrary, the points all fall very nearly in the curve, we are justified in assuming the essential accuracy of the data. Judged by this standard, the rainfall and runoff data of Hudson and Genesee rivers, as given below, are exceedingly accurate. Indeed, from this point of view, they give, so far as the author is aware, more nearly the true values than any extended set of gagings from so large areas thus far made. The following are the rainfalls and runoffs of Hudson river for the 11 years from 1888–98, inclusive, arranged in the order of the magnitude of the rainfall :

HUDSON RIVER.

Year.	Rainfall.	Runoff.
1892	53.87	33.08
1890	50.35	28.56
1898	48.30	27.65
1897	46.51	26.19
1896	45.21	23.65
1888	43.92	23.64
1891	42.96[1]	20.56
1889	42.96[1]	21.71
1893	42.18	21.91
1894	41.37	19.37
1895	36.67	17.46

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[1] Probably 42.96 inches rainfall for 1889 is only app.oximately correct. Data incomplete.

For Genesee river these data take the following form :

GENESEE RIVER.

Year.	Rainfall.	Runoff.
1894	47.79	19.38
1890	47.54	21.22
1898	42.50	15.13
1892	41.69	15.42
1896	40.68	13.35
1893	39.30	13.35
1891	38.12	14.05
1897	34.39	9.38
1895	31.00	6.67

Comparison of these two series will show that Hudson river data are somewhat better than Genesee. In explanation of the foregoing tables, it may be remarked that continuous daily gagings of the Hudson river have been made at Mechanicville, where the drainage area is 4,500 square miles, from October, 1887, to the present time. It is from these gagings that the Hudson river data have been derived.

Gagings of Oatka creek, a tributary of Genesee river, were made at Warsaw, New York, where the drainage area is 27.5 square miles, from April, 1890, to December, 1892. The Genesee river itself was gaged at Mount Morris, where the drainage area is 1,070 square miles, from September, 1893, to November, 1896, inclusive. Gagings of Genesee river have also been made at Rochester, where the drainage area is taken at 2,365 square miles, from March, 1893, to the present time. It is from these several gagings of Genesee river and its tributary Oatka creek, that the herein given data of Genesee river runoff are derived.

By way of showing the probable accuracy of these data, we may consider the following tables applying to Hudson and Genesee rivers :[1]

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[1] The data embodied in these tables have been partially given in a paper On the Application of the Principles of Forestry and Water Storage to the Mill Streams of New York State, read before the American Paper and Pulp Association at its annual meeting, February, 1899. Since preparing that paper, the author has been able to extend and correct the data somewhat.

HUDSON RIVER.

Year.	Rainfall.	Variation from the Mean.	Runoff.	Variation from the Mean.	Evapora'n.	Variation from the Mean.
1888	43.92	—1.02	23.64	—0.34	20.28	—0.73
1889	42.96	—1.90	21.71	—2.27	21.25	+0.24
1890	50.35	+5.41	28.56	+4.58	21.79	+0.78
1891	42.96	—1.98	20.56	—3.42	22.40	+1.39
1892	53.87	+8.93	33.08	+9.10	20.79	—0.22
1893	42.18	—2.76	21.91	—2.07	20.27	—0.74
1894	41.37	—3.57	19.37	—4.61	22.60	+1.59
1895	36.67	—8.27	17.46	—6.52	19.21	—1.80
1896	45.21	+0.27	23.63	—0.35	21.58	+0.57
1897	46.51	+1.57	26.19	+2.21	20.32	—0.69
1898	48.30	+3.36	27.65	+3.67	20.65	—0.36
Means	44.94	—19.58	23.98	—19.58	21.01	+4.57
		+19.54		+19.56		—4.54
		—0.04		—0.02		+0.03

GENESEE RIVER.

Year.	Rainfall.	Variation from the Mean.	Runoff.	Variation from the Mean.	Evapora'n.	Variation from the Mean.
1890	47.54	+7.21	21.22	+7.06	26.32	+0.14
1891	38.12	—2.21	14.05	—0.11	24.07	—2.11
1892	41.69	+1.36	15.42	+1.26	26.27	—0.09
1893	39.30	—1.03	13.35	—0.81	25.95	—0.23
1894	47.79	+7.46	19.38	+5.22	28.41	+2.23
1895	31.00	—9.33	6.67	—7.49	24.33	—1.85
1896	40.68	+0.35	12.80	—1.36	27.88	+1.70
1897	34.39	—5.94	9.38	—4.78	25.01	—1.17
1898	42.50	+2.17	15.13	+0.97	27.37	+1.19
Means	40.33	+18.55	14.16	—14.55	26.18	—5.36
		—18.51		+14.51		+5.35
		+0.04		—0.04		—0.01

On comparing these two tables, it appears clear that the water yields of these two streams are vastly different. In searching for a reason for the great difference which appears, the author is disposed to assign as a principal cause, the difference in forestration, the Hudson area being still largely in forest, while the Genesee is almost totally deforested, and under high cultivation, either for grain farming or grazing. Space will not permit of discussing the preliminary data already given, in its application to these two drainage areas, and the author, therefore, contents himself with offering the suggestion that with the data now well before you, no more prolific subject for a thesis could be selected than to apply these data to an exhaustive discussion of these two catchment basins, for which there is now enough runoff data to permit of fairly exhaustive discussion. It is hoped that some of the young gentlemen of the classes here represented, may find this a useful suggestion.

By way of comparing the rainfalls, runoffs and evaporation of these two drainage areas, the following tables have been prepared :

COMPARISON OF RAINFALLS.

Year.	Rainfall of Hudson Basin.	Rainfall of Genesee Basin.	Difference.
1890	50.35	47.54	+2.81
1891	42.96	38.12	+4.84
1892	53.87	41.69	+12.18
1893	42.18	39.30	+2.88
1894	41.37	47.79	—6.42
1895	36.67	31.00	+5.67
1896	45.21	40.68	+4.53
1897	46.51	34.39	+12.12
1898	48.30	42.50	+5.80
Means	45.27	40.33	4.94

COMPARISON OF RUNOFFS.

Year.	Runoff of Hudson Basin.	Runoff of Genesee Basin.	Difference.
1890	28.56	21.22	7.34
1891	20.56	14.05	6.51
1892	33.08	15.42	17.66
1893	21.91	33.35	8.56
1894	19.37	19.38	—0.01
1895	17.46	6.67	10.79
1896	23.63	12.80	10.83
1897	26.19	9.38	16.81
1898	27.65	15.13	12.52
Means	24.27	14.16	10.11

COMPARISON OF EVAPORATION.

Year.	Evaporation of Hudson Basin.	Evaporation of Genesee Basin.	Difference.
1890	21.79	26.32	4.53
1891	22.40	24.07	1.67
1892	20.79	26.27	5.48
1893	20.27	25.95	5.68
1894	22.60	28.41	5.81
1895	19.21	24.33	5.12
1896	21.58	27.88	6.30
1897	20.32	25.01	4.69
1898	20.65	27.37	6.72
Means	21.07	26.18	5.11

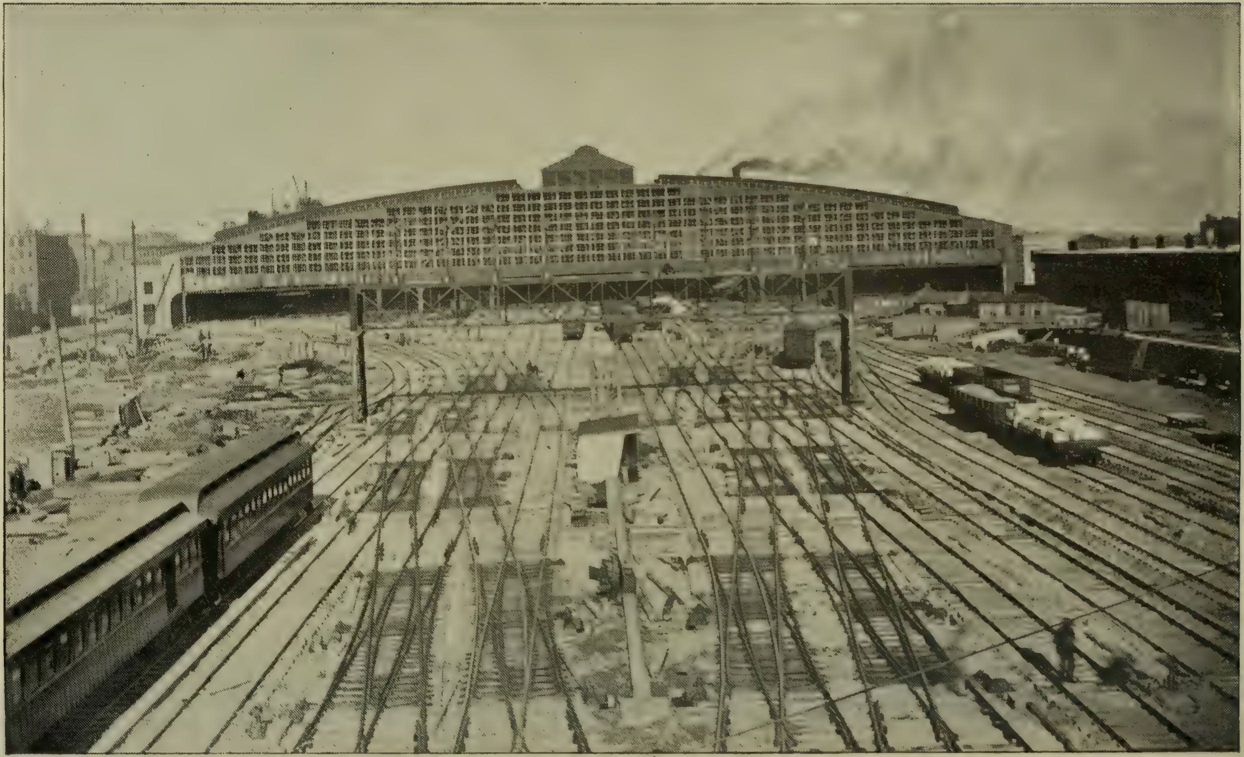
In September, 1897, the author presented a popular paper On Stream Flow in Relation to Forests, before the American Forestry Association at its Nashville meeting. Some elaboration of the data not given in this paper may be found there. Indeed that paper may be especially consulted for an illustration of practical use of such data as is herein included.

In conclusion the deduction may be tentatively drawn from the foregoing data that hardwood forests may consume from about five to ten inches in depth of water over the ground area in each growing season. Probably from 6 to 8 inches is a fair average for deciduous forests although broad leaved forests may consume somewhat more. The most of the evergreens are small water consumers, the larch being, however, an exception. Spruce and pine forests apparently require only a few inches on the ground area per year. In order to insure enough we may allow them from 4 to 6 inches per year.

The figures show therefore that the kind of forest must be taken into account in estimating the effect on yield of streams.

As regards cultivated crops the figures show demands for about 12 to 15 to 18 inches on the ground area for cereals and grass crops. For vineyards, and hop yards the tabulated figures may be increased by at least 50 per cent. to cover evaporation from the naked soil between the plants, thus giving about 8 to 11 inches for these two agricultural crops. Everything goes to show that for grass crops the consumption of water is very large, clover transpiring from 13.6 inches to 18.0 inches, and grass crops even more than this. As shown by the experiments of Baldwin Latham, Italian ray-grass will, when given a full water supply, use up in one year several times the average annual rainfall of this state.

As a final tentative proposition we may say therefore that highly cultivated farming areas will consume in surface evaporation and plant transpiration from two to three times as much water as average deciduous forests, and from three to five times as much as average evergreen forests. In mixed forests the water consumption will depend upon the relative proportion of the different kind of trees.



BOSTON'S GREAT UNION DEPOT: TRAIN SHED FROM THE TOWER.

[By permission of the *Buffalo Express*.]

MAINTENANCE OF WAY.

BY WILLIAM M. TORRANCE, C.E. '95, MEM. C. E. CLUB, CLEVELAND, O.

To a large extent, the modern railway depends for its success as a business enterprise, on its Maintenance of Way department. On the best railways this department is under the supervision of civil engineers of established reputation and great ability whose business it is to see that the railway company's tracks, buildings, bridges, etc., are properly maintained, or as nearly so as may be done with the funds placed at their disposal by the managers of the road.

The civil engineers in charge of the maintenance of the road must not only be good civil engineers capable of solving mathematical problems peculiar to such work, but they must be men of tact and judgment, in order to successfully carry out the wishes of their superiors with the raw material—a limited supply of money and laborers—at their disposal.

It is not the intention of this article to go into the business details of a Maintenance of Way department office, but the writer will present a few of the engineering problems such as need attention almost daily in the successful operation of maintenance work. In the case of nearly all railroads built in the early days of railroading in this country, the preliminary construction was such as to satisfy merely the needs of that time with no forethought as to the future. The rolling stock of the road consisted of light locomotives and cars, and the general construction of the road was correspondingly light. Now, in order to successfully carry on the increased business of the road, heavier locomotives and cars are being used than could have been contemplated by the original builders. The speed of trains has also increased greatly during the past few years. These causes render it necessary for the Maintenance of Way department to rebuild the roadbed of better material than was first used, by substituting heavier rails and first-class ballast. New bridges must be built in many places to carry the increased loads, and constant watchfulness must at all times be used to locate that part of the property of the railway that needs the most prompt attemtion, so that the available funds of the department may be expended in the manner best calculated to render the traffic of the road free from accidents and to make all the property of the road present the best possible appearance.

In properly maintaining the property of the railway, the tracks of road furnish naturally the chief source of employment for the maintenance of way department. When renewed, they are built in the best manner possible, with modern heavy rail sections and modern switches and frogs, and the arrangements of the yards at the various stations are made matters of scientific study. When the main line is being improved by replacing rails with heavier sections and the new ballast and ties are being put in, the engineer sees to it that the finished road will have the best possible grade line and alignment. Preliminary profiles and tangent lines are run and both line and grade stakes, on established lines and grades, are given the track men for all that portion of the road—both on tangents and curves—then being improved.

Following are a few of the problems, similar to actual ones in maintenance work that have occurred during the writer's experience as Assistant Engineer Maintenance of Way of the Cleveland-Indianapolis Division of the C. C. C. & St. L. Ry.

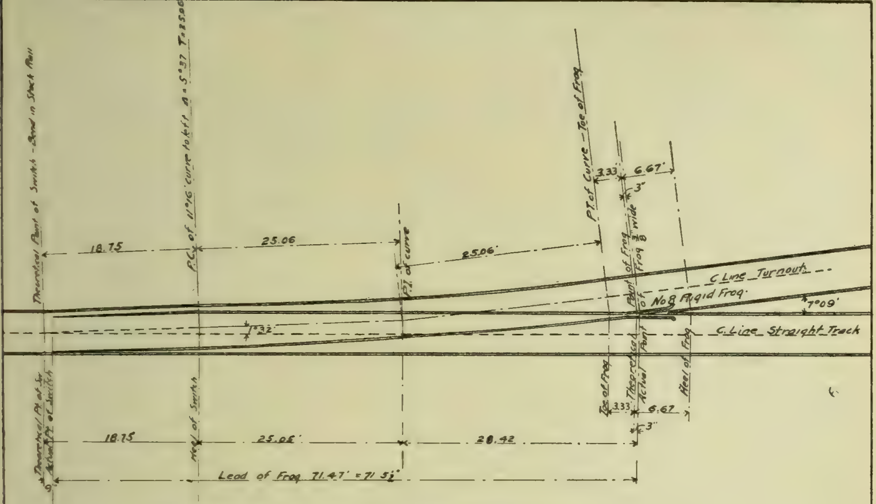
TURNOUTS.

A turnout consists of that portion of the track where one track leads out from another. In Fig. 1 is shown a turnout from straight track for a No. 8 rigid frog with split switch. The lead of the frog (the distance from point of switch to point of frog measured in a direction parallel with the straight track) shown is, if the turnout be constructed so that the curvature of the lead track is a minimum, 71.47 ft. Herewith is the algebraic work from which this distance is obtained.

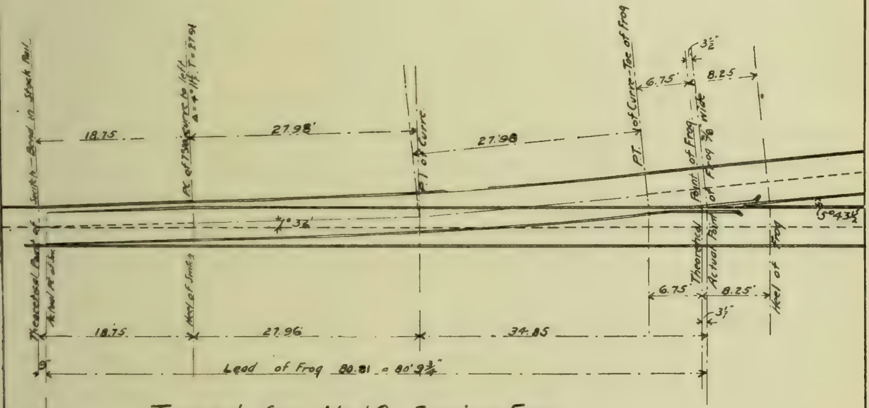
The switch rail shown is 18 ft. in length with its point ¼ in. wide and with its gauge line 6 in. from the gauge line of the other rail at its heel. The frog shown is 10 ft. long, the theoretical point (the point where the two gauge lines of the frog intersect each other) being 3.33 ft. from the toe and 6.67 ft. from the heel of the frog. The actual, ⅜ in. wide, point of the frog is 3 in. toward the heel from the theoretical point. The gauge line of the switch rail and the gauge lines of the frog are straight lines, and the problem becomes that of determining the proper point of intersection of the tangent at the switch with the tangent at the frog, in order to make the curvature uniform between the heel of switch and toe of frog.

The angle of switch tangent with straight track tangent =

$$\sin^{-1}\frac{.50}{18.75} = 1^\circ\ 32'.$$



Turnout for No. 8 Rigid Frog.
Fig. 1.



Turnout for No. 10 Spring Frog
Fig 2.

The angle of frog tangent = angle of No. 8 frog = 7° 09'.

Frog tangent is distant from tangent of straight track at toe of frog 4.71 ft. (distance between gauge lines of either track = distance between center lines of track at theoretical frog point) less [3.33 ft. sin 7° 09' = .42 ft.] = 4.29 ft.

Switch tangent is distant .50 ft. from straight track tangent at heel of switch.

Therefore distance normal to straight track between switch tangent at heel of switch and frog tangent at toe of frog

$= 4.29 - .50 = 3.79$ ft.
$=$ tangent distances of curve $[\sin 1° 32' + \sin 7° 09']$
$=$ " " " " $[.02676 + .12447]$.

From which we obtain

Tangent distance $= 379 \div .15123 = 25.06$ ft.

Therefore the lead of the frog = distance between actual points of switch and frog

$= [18 + 25.06] \cos 1° 32' + [25.06 + 3.58] \cos 7° 09'$
$= 43.05 + 28.42$
$= 71.47$ ft.

The curvature in the turnout is found from the data we now have to be that of an 11° 16′ curve.

Fig. 2 shows a turnout for No. 10 spring frog with split switch. The proper lead of this frog, following the same method as with No. 8 rigid frog, is 80.81 ft.

It is evident that, if a lead be used shorter than the theoretical lead as above figured, the curvature of the turnout will be increased toward the switch, and if a lead longer than the theoretical lead be used the curvature will be increased toward the frog.

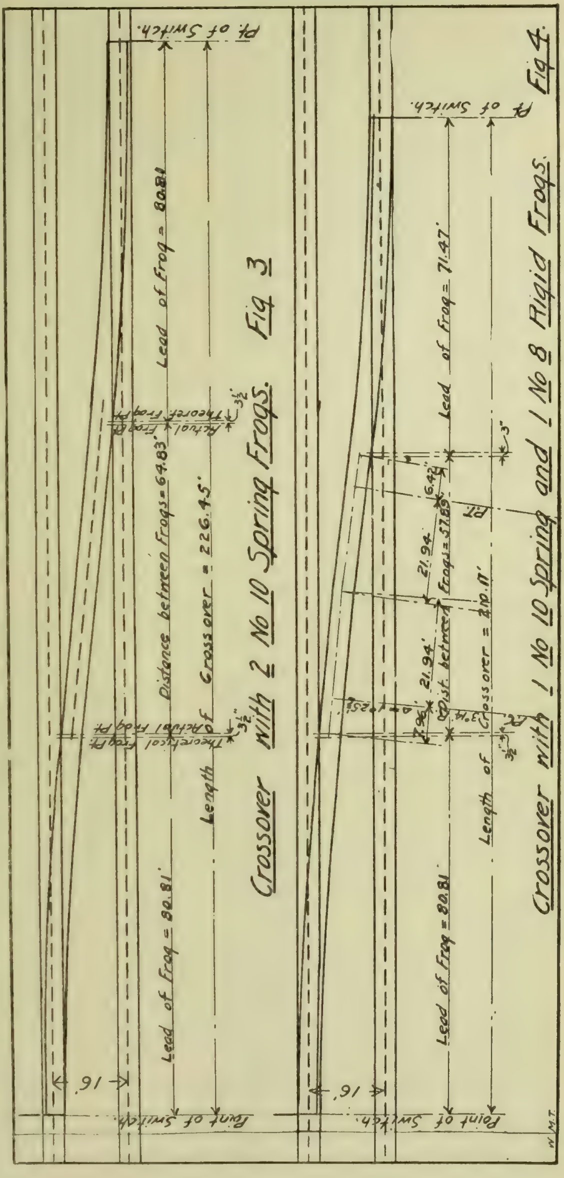
It is also evident that the proper leads for frogs depend on the lengths of the frogs and switches used as well as on the distance at which the heel of the switch is set from the other rail.

The proper lead for a turnout leading from a curved track is an entirely different problem, as the switch rail will most likely be curved. If the direction of the tangent at the heel of the switch be known, however, the lead of the frog can be determined. It is, however, a difficult problem, requiring much algebraic and trigonometric work.

In setting the stakes for turnouts it is usual to merely mark the center line of the track at the actual point of switch and the center lines of each track at the actual point of frog. The trackmen will then put the switch and frogs in positions thus indicated and put in the remainder of the turnout without stakes. If greater accuracy of alignment through the turnout than this method affords is desired, the curve between the switch and the frog can be staked out.

CROSSOVERS.

A crossover is a short piece of track connecting two parallel tracks and consists of two turnouts and a short piece of track between the frogs of the same.


Pt. of Switch.
Lead of Frog = 80.81
Distance between Frogs = 64.83'
Length of Crossover = 226.45'
Theoret. Frog Pt.
Actual Frog Pt.
3½"
Lead of Frog = 80.81'
Point of Switch.
16'
Crossover with 2 No 10 Spring Frogs. Fig 3
Pt. of Switch.
Lead of Frog = 71.47'
16.42'
Dist. between Frogs = 57.85'
21.94'
21.94'
Length of Crossover = 210.11'
Pt.
3"
3½"
Lead of Frog = 80.81
Point of Switch.
16'
Crossover with 1 No 10 Spring and 1 No 8 Rigid Frogs. Fig 4.

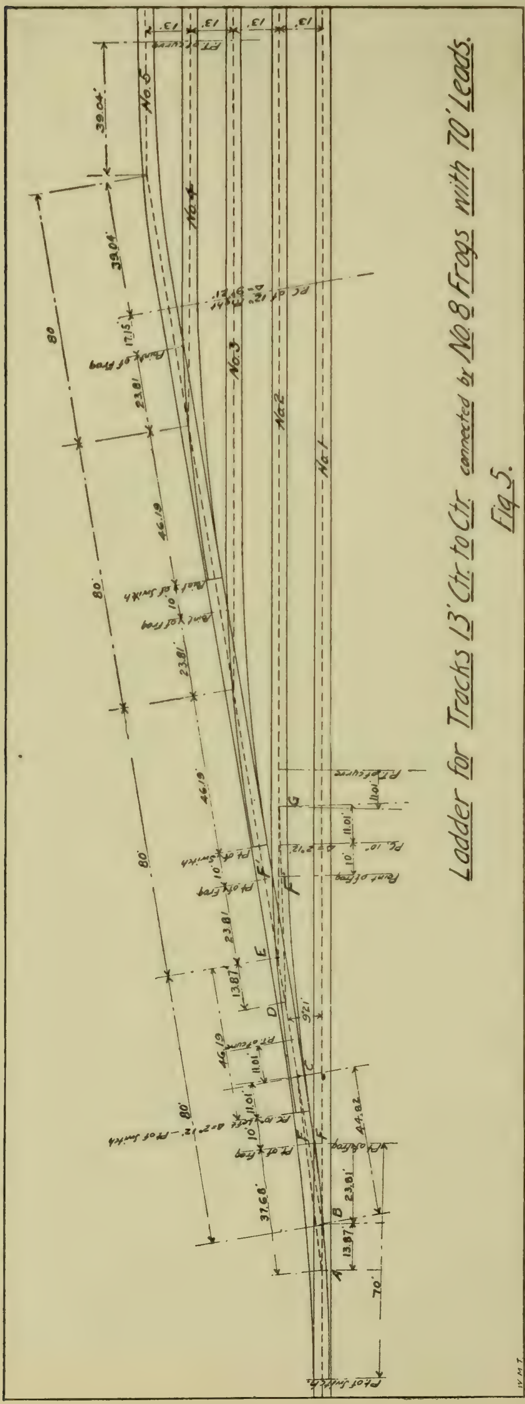
The ordinary rule for determining the length of track between the two frog points of a crossover is to substract two times the gauge of the track from the distance, center to center of tracks, and multiply by the index number of the frog used.

Fig. 3 shows a crossover between two tracks 16 ft. center to center, No. 10 spring frogs being used. The distance between the frog points by the rule given above would be —(16—9.42) × 10 = 65.8′.

This distance is probably all right enough for practical use in some cases but, strictly speaking, the proper distance between the frog points should be 64.83′. This distance is obtained as follows :

The actual distance from the center line of the crossover track to the center line of either parallel track at the actual point of frog is—

$\frac{4.71}{2} + \left(\frac{4.71}{2} \div \cos 5^\circ\ 43\tfrac{1}{2}'\right) + \frac{3}{8}'' = 4.75'$ and therefore the distance normal to the direction of the parallel tracks between the points on the


Ladder for Tracks 13' Ctr. to Ctr. connected by No. 8 Frogs with 70' Leads.
Fig. 5.

center line of the crossover track opposite the two frog points is 16 — (2 × 4.75) = 6.50.

The angle of a No. 10 frog is 5°43½' and the tangent of this angle is .10026

6.50 ÷ .10026 = 64.83' = proper distance, measured on line parallel with direction of parallel tracks, between actual frog points of crossover. Using the lead for No. 10 frog as 80.81' as heretofore determined and shown in Fig. 2, the total length of crossover is 226.45'.

Fig. 4 shows a crossover between two tracks 16 ft. center to center, the lead from one track being that of a No. 10 spring frog and the lead from the other that of a No. 8 rigid frog

The ordinary rule for determining the length of track between the two frogs in such a case as this is to multiply the distance center to center of tracks less two times gauge distance (4.71 ft.) by the half sum of the two frog numbers. This rule would give for the distance between frogs (16 — 9.42) × 9 = 59.22 ft.

With frogs as shown, this distance should be 57.89 ft. which is deduced as shown on the drawing by making the curvature uniform between the heels of the respective frogs.

In laying out crossovers in any yard, especially if the parallel tracks are close together, care is taken to have the length staked out strictly correct as otherwise the alignment will be poor.

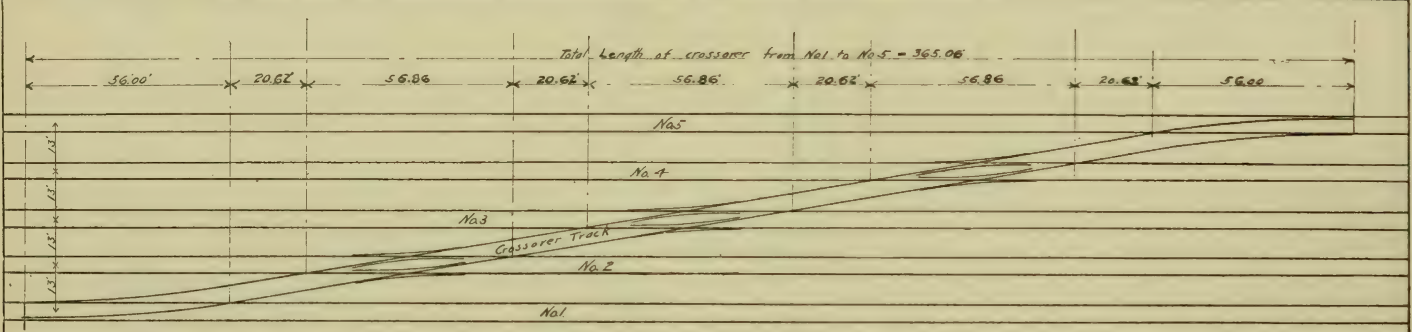
Fig. 7 shows a yard of 5 parallel tracks, 13 ft. center to center. Suppose it becomes necessary to lay out in this yard a set of crossovers such that it will be possible for train movements from any track to any other track shown above it, if movement be toward the right, and from any track to any other track shown below it, if movement be toward the left.

Suppose also that it is desired to accomplish these possibilities in the shortest possible longitudinal space of track. To accomplish this with ordinary track material No. 6 frogs are used, as shown in this Fig. 7. A customary lead for a No. 6 frog is 56 ft. which in any actual case varies from this figure according to results obtained in the same manner as the No. 8 rigid frog lead found above, and the total length for the crossover between each two adjacent tracks is found to be 132.62 ft. To obtain good track, switch points, facing in opposite directions should be at least 10 ft. apart and the arrangement shown in the figure contemplates the use of this distance between the ends of consecutive crossovers.

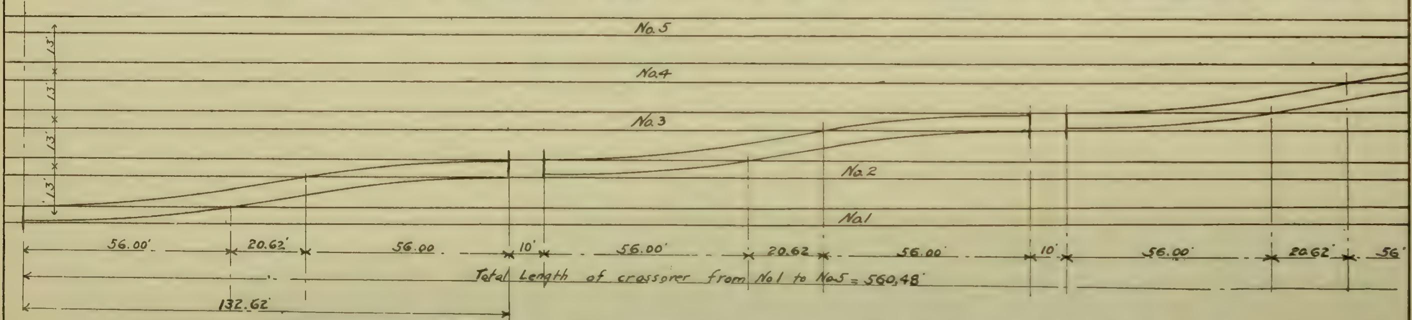
The total length of crossover to get from track No. 1 to track No. 5 by this arrangement is 560.48 ft.

Fig. 6 shows an arrangement of tracks to accomplish the same objects in a much shorter space using, however, the same frog angle as used in the arrangement just discussed. The peculiar feature of this arrangement is the combination slip switch and crossing frogs. These combinations are a comparatively new appliance for track work but already they are being used extensively. In the case of arrangement shown on Fig. 7 and Fig. 6, it can be readily seen that the arrangement shown on Fig. 6 is much better both for distance saved in the total length of crossover for the whole yard—nearly 200 ft.—and in alignment for this crossover. For if a train were to pass from track No. 1 toward the right to track No. 4 or 5 by arrangement shown in Fig. 7 the movement would be very irregular and therefore necessarily slow on account of the many changes in direction of curvature of the crossover ; while if arrangement shown in Fig. 6 be used there are only two curves to traverse in any case.

The arrangement of tracks near the end of a large modern passenger station must be such that a train coming toward the station, from



Crossovers for Yard of 5 parallel straight tracks using No 6 Frogs and No 6 Combinations. Fig. 6.



Crossovers for Yard of 5 parallel straight tracks using No 6 Frogs. Fig. 7.

any of the tracks leading to the same, can have the option, as it were of any one of several tracks in the station.

It will nearly always be desired in such cases to obtain this option in the shortest possible longitudinal space of track from the end of the station, and the crossover arrangements for passing a train from one track to another must be of the shortest possible length. The accompanying cut, taken from an article in the issue of Jan. 1 of the Illustrated Buffalo Express, shows one end of the new Boston Union Station and in the foreground are seen a number of these combination slip switch and crossing frogs in actual use for accomplishing the purpose outlined above. The writer has had no opportunity of visiting Boston since the erection of this station, but it is probable that the arrangement of crossover combinations is continued back toward the point of view of the picture so that there are many tracks from which approaching trains could pass to any track in the station trainshed.

One of the many good points about the use of these combinations, is the fact that a train passing over one of them can not run through any of the four switches comprised, as all four are operated by one switch lever and the tracks passing through must either be set for movements straight ahead in all cases or for movements to turn out in all cases.

TRACK LADDERS.

By a track ladder is usually meant a track connecting the ends of several parallel tracks in the manner shown in Fig. 5. Some such an arrangement is always necessary at the end of a switching yard in which trains are made up or divided.

Fig. 5 shows a detailed design of a ladder for tracks 13 ft. centers, being connected to the ladder track by No. 8 frogs, having 70 ft. leads and the ladder track itself being connected to the track designated as No. 1 by the same kind of a lead. This is a very common sort of a yard and the writer believes, from his own experience, that this sort of ladder is as good as could be obtained, as it leads out from the track designated by No. 1 at as sharp an angle as possible consistent with good track work on switches and frogs (this angle is 9° 21′ which is the anti-sine of 13 ft., the distance center to center of tracks, divided by 80 ft., the distance between consecutive frogs on the ladder) and thus allows the least possible amount of dead track [track upon which it is not possible to stand cars and allow other cars to pass on the other tracks] consistent with good track work.

The writer has in mind one of the Cleveland yards of one of the large railway companies entering this city, in which the ladders are made up of No. 10 frogs and laid at the angle of that frog (5° 43½′) with the 10 or 15 parallel tracks comprising the yard. This makes the distance, center to center of consecutive frogs along the ladder, about ten times the distance center to center of parallel tracks or about 130 ft. and though there is much less curvature in the tracks (being only that of the turnouts themselves) much valuable land is wasted and much useless dead track exists in the ladder itself.

The problem of designing the ladder shown in Fig. 5, is practically that of solving the triangle *A B C* and the equal but reversed triangle *D E G*. In these triangles the points *B* and *E* are intersecting points of the center lines of the tracks of the yard with the ladder track. The angles of these triangles are respectively; supplement of 9° 21′, the angle of the ladder; 7° 09′, the angle of No. 8 frog; and 2° 12′, the central angle of the curve beyond the turnout in each case. The points *A* and *D* are at the intersections of the center lines of the tracks at the frog in each case, and the distance *A F* or *D F′* is:

$$\frac{4.71 \times \sin 86^\circ\ 25\tfrac{1}{2}'}{\sin 7^\circ\ 09'} = 37.68 \text{ ft.}$$

The distance *F C* or *F′ G* = 10 ft., the length of straight track allowed from the point of the frog towards it heel + 11.01′, the tangent distance of the 2° 12′ of 10° curve = 21.01′.

Therefore the side of the triangle *A C* or *D G* = 58.69 ft. From which we find the sides *A B* and *B C* or *D E* and *E G* to be respectively 13.87 ft. and 44.82 ft. This gives us the distance *B F* or *E F′*, which is 23.81 ft. and therefore enough data to determine the position for all frogs and switches from the point *B*, the point of intersection of the center line of the ladder with the center line of the No. 1 track.

In designing ladder tracks, it is not always possible to choose the angle of intersection of the same with the tracks it connects. It is often necessary to use a much larger angle than that shown in Fig. 5 in which case each consecutive two tracks or even three tracks must be connected before joining the ladder track, as there will not be room enough on the ladder track for turnouts from the same to each individual track. The writer at one time designed, set stakes for, and oversaw the construction of a yard of 15 tracks in which the ladder track made an angle of some 33° with the yard, the consecutive tracks being 18 ft. center to center, and the design such that each three consecutive tracks were connected by No. 8 frog leads of 70 ft. each, and thus only 5 tracks were directly connected to the ladder track by leads of the same kind as given above, the distance between consecutive frogs along the ladder, being about 95 ft., all turnouts being from straight track and the curvature outside the turnouts in all cases being that of 10° curves.

TRACK ALLIGNMENT.

The engineer maintenance of way of any railroad will usually find that the allignment can be materially improved at many points along the road. This he will find to be true of both tangents and curves. The eye is a bad judge of a true straight line or of a regular curve, and no matter how perfect the alignment might have been at the time of construction of the road, and no matter how much time trackmen may have spent trying to improve the line, it will become far from perfect in the course of a few years. Usually, however, the line never was perfect, and especially is this the case on long tangents which more than likely were originally run in a hurry and with crude instruments. It will also be found in most cases that no transition curves were ever put in at the ends of the curves.

In relining track on any section of the road, the line to be established and staked out should be governed by either the tangents or the curves, depending on the predominance of the one or the other. In either case the track should be surveyed and mapped so that it can be readily seen in just what shape it exists at the outset, and so that on the map so made a better line can be established for the surveyor to stake out for use of trackmen in the actual work of relining the track. In case the curves predominate, each curve or set of curves is considered separately, and the best arrangement consistent with allowable expense is determined. Sometimes the whole allignment for two or three miles is changed so as to decrease the number of curves, and also to reduce the maximum degree of curvature to be found on same. This is often done where the original road followed along the bed of a stream and where the location first chosen for the road followed the winding of the same. In such cases there will be found many reverse curves and many quite sharp curves. By carefully mapping such a piece of track, or by actual observance on the ground, a new line may be found possible which will, at the expense of a few hundred dollars for grading and extra right of way, rid the road of many bad curves.

In case the tangents predominate, each tangent is taken separately. A preliminary straight line is run from one end to the other and offsets to this line, from the center line of the track at intervals of say 500 ft.

are measured. This line can then be plotted as a straight line on a piece of profile paper, and the position in which the track was found, relative to it, can be plotted to a greatly exaggerated scale by the offsets taken. By means of this plat it can readily be determined what is the most desirable thing to do in establishing the line for the tangent. Often it is a difficult matter to run the preliminary straight line along the tangent. When possible it is run from one end continuously to a constant foresight at the other. Where the intervening distance, or where an intervening rise in the grade, prevents this, the best plan is to range in with the transit between foresights at either end at a point midway between the ends, or at the top of the rise in grade and, from the point thus found, run the straight line to either end. The best of weather is necessary for this kind of work, clear days in the coldest winter weather being preferable. On one occasion, in this kind of weather, the writer placed a foresight above the top of a through bridge near one end of a tangent, 8 miles east from a summit of grade, and 14 miles east from the west end of the tangent, at which point also a large foresight was erected. The instrument was then set up at the summit of the grade mentioned. The foresights could be plainly seen and two preliminary lines were run, one each way from this summit to the ends of the tangent.

In the case just cited it was impossible to range in at the summit, between the foresights at the ends, as there was an angle of some minutes in the track at this summit, which would make the ranging in point outside the cleared right of way of the road. This angle was carefully measured, however, for purpose of plotting.

After all the tangents in this section of the road have been determined as above, curves as desired can be run in between the same, care being taken to run in the proper transition curves at the ends and to properly indicate for use of the trackmen just where the ends of these transition curves are found and what elevation is desired on the curve between same, the outer ends of the transition curves being points of no elevation. A customary rule for determining the length of a transition curve is to have the elevation change ½ in. in each rail length of 30 ft. so that it will have a length of 60 ft. times the number of inches elevation desired in the track, on the curve.

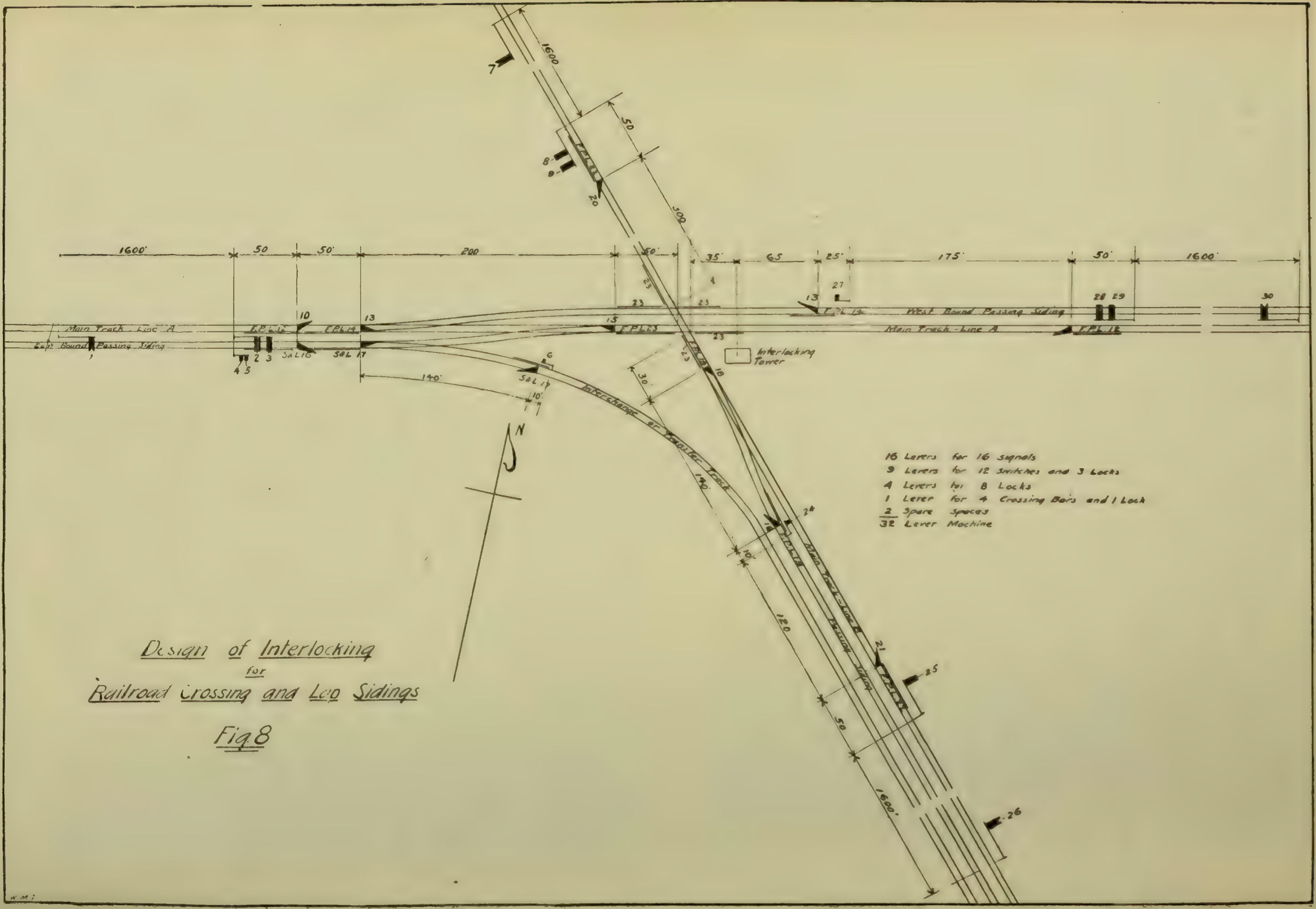
In relining track it is always desirable, of course, to vary as little as possible from the existing line and there' will always be governing points, such as crossings with other railways, station platforms, etc., at which it is undesirable to make the slightest change. When such governing points come on long curves, a rather difficult problem confronts the engineer. The curve is carefully surveyed and notes sufficient to locate points on track center line at short intervals, say each 100 ft., along the whole length of the curve are taken. Then the position of the point of intersection of the curve is calculated (assuming that it cannot be obtained on the ground) and with one tangent as a base line, the points on the existing track center are located by latitudes and departures from the point of intersection and this base line. Then various curves can be tried on these tangents and the latitudes and departures of points on the same can be figured and compared with the latitudes and departures of the same points as existing in the track as found. A few trials will suffice to determine the proper curve to fit the desired conditions and data for running in the curves can be compiled from existing notes.

The writer recalls a curve of some 92° central angle, the curvature being about that of a 5° curve which he was required to relign in such a manner that the throw should be very little at any point and none at all at a crossing with another railway that occurred on the curve. He was able by the method outlined above to accomplish these objects with little trouble.

INTERLOCKING.

Fig. 8 shows an arrangement for interlocking a crossing such as a good maintenance engineer is able to design. We will suppose that the engineer enployed for the east and west line marked line "*A*" finds here a crossing with another road in which heavy traffic is maintained and that he finds on his own line a passing siding, marked east bound passing siding, on the drawing. He also finds that the other road has a passing siding leading out toward the south from the crossing. The crossing is attended to by a watchman who is provided with an arrangement for displaying a danger signal to either road or both roads if desired. This watchman's duties also include the tending of the switches immediately at the crossing. Under this arrangement, high speed for either line is dangerous, as the signal can not be seen at any great distance and the faithfulness of the watchman is in any case a question of much doubt.

The engineer finds also the need of additional passing siding accommodations at this point. Therefore while providing an interlocking arrangement for protecting the crossing, he also provides additional passing siding track in the manner known as a "Lap Siding" which arrangement will allow two trains going in opposite directions to wait



Design of Interlocking for Railroad Crossing and Lap Sidings

Fig. 8

at the crossing for a third train to pass in either direction on the main track and then allow both to get on main track and go on their way without waiting for each other and without any delay in closing switches behind them, as they are manipulated from the interlocking tower.

The proper designing of the interlocking for such an arrangement as shown requires considerable study on the part of the maintenance engineer. Trains coming from any direction, must be protected, and the number of levers in the interlocking machine must be reduced to a minimum to save expense in installing the machinery.

In order to render the traffic perfectly safe derailing switches are put into each track on which movements of trains could interfere with those on any other track and the design contemplates that all switch locks, switches derails and detector bars be so connected with the interlocking machine in the tower that it will be impossible for accidents to happen at the crossing, unless the accident be that of a train running against signals—and hence off a derailing switch.

The design of the machine is to be such also that train movements that do not interfere with each other can be performed at the same time,—as for example, movements from the west bound passing siding, west, and from the east bound passing siding, east, do not interfere with each other and the machine must be designed so that the locks, switches and signals governing these two movements can be set so as to allow both to take place at once.

Below is presented a synopsis of the arrangement of the levers of the machine such as to set the various locks. switches, signals, etc., operated by the machine for train movements in any direction. On the drawing these locks, switches, signals, etc., have been numbered by numbers from 1 to 30. We will suppose these numbers to represent the numbers of the levers in the machine which operates the movements numbered, although in actually installing the interlocking, this numbering would no doubt be changed for sake of simplicity of arrangement of locking device in the machine. The drawing shows the normal positions for all devices—the signals all being set vertical, *i. e.*, danger position, the derailing switches all being set for derailment, and in the tower all levers of the machine being thrown back.

LEVER MOVEMENTS.

No. 1. To change the machine from normal position to allow train movements on main track line "A" toward the east.

Lever No. 23, to throw crossing bars to make sure no cars are standing on either track at the crossing.

Lever No. 12, to unlock the derailing switches, east and west on main track.

Levers No. 10 and No. 11, to close these derails.

Lever No. 3, to signal the train that track is set for it to pass.

Lever No. 1, to serve the same purpose as lever No. 3, but the signal No. 1, being at a greater distance from the crossing than signal No. 3, is necessary in order to maintain high speed, as the locomotive driver can see it at a much greater distance from the crossing.

No. 2. To change from normal position to allow train movement on main track line "A" toward the west.

Levers Nos. 23, 12, 10 and 11 as before.

Levers Nos. 28 and 30, to signal the train that track is set for it to pass.

If the track had previously been set for movements on main track east as given under No. 1, to obtain position No. 2 it would only be necessary to throw back levers No. 1 and No. 3 and pull out levers No. 28 and No. 30, the setting for the track being the same in both cases. Of course the locking of the machine is such as to prevent levers 28 and 30 from being pulled out to signal a train from the east until levers Nos. 3 and 1 are thrown back to normal to prevent signaling trains from the west.

No. 3. To change from normal position to allow train movements on main track line "B" toward the south.

Lever 23, for crossing bars.

Lever 22, for locks at derailing switches.

Levers 20 and 21, to close derails.

Levers 9 and 7, to set signals for train to pass.

No 4. To change from normal position to allow train movements on main track line "B", toward the north.

Levers 23, 22 20 and 21 as before.

Levers 25 and 26 to signal train to pass.

No. 5. To change from normal position to allow train movements from east bound passing siding, east to main track, Line "A".

Lever 15, to set switch to siding.

Levers 12 and 11 to close derail east of crossing.

Lever 16, to unlock and close derailing switch on siding.

Lever 5 to signal train to pass.

No. 6. To change from normal position to allow train movements from main track Line "A" west to east bound passing siding.

Levers 23, 15, 12, 11, and 16 as before.

Lever 29, to signal train to pass.

No. 7. To change from normal position to allow train movements from east bound passing siding Line "A" to interchange track.

Lever 17, to unlock and set derail on interchange track and switch from siding to same.

Lever 16, to unlock and set derail on passing siding.

Lever 4 to signal train to pass.

No. 8. To change from normal to allow train movements from interchange track to east bound passing siding, Line "A".

Levers 17 and 16 as before.

Lever 6, to signal train to pass.

It is obvious that the locking should be so arranged in the interlocking machine that it will be possible to pull out lever 16 after pulling out lever 17 so that either the 7th or 8th movements just mentioned can take place while movements are taking place on either main track or on the west bound passing siding Line "A".

No. 9. To change from normal to allow train movements from west bound passing siding, west to main track, Line "A".

Levers 23, 12, 10, 14 and 13 in order named to set the track.

Lever 27 to signal the train to pass.

No. 10. To change from normal to allow train movements from main track Line "A" east to west bound passing siding.

Levers 23, 12, 10, 14 and 13, in order named.

Lever 2 to signal train to pass.

No. 11. To change from normal position to allow train movements from main track, line "B", south to passing siding.

Levers 23, 22, 20, 19 and 18 in order named, to set the track.

Lever 8 to signal the train to pass.

No. 12. To change from normal position to allow train movements from passing siding, line "B", north to main track.

Levers, 23, 22, 20, 19 and 18 as before.

Lever 24 to signal train to pass.

The matter of proper signaling for railroad crossings, as well as at other places, is of growing importance on nearly all roads, and for any important road requires for its management men of peculiar ability and experience in designing, installing, and operating these appliances for the safety of high speed traffic.

On many roads the interlocking and other signaling for the road is looked after by a separate department, at the head of which is the signal engineer.

BRIDGES.

The management of any railway will always depend on its maintenance of way department for the care of the bridges of the road. The engineers in charge of this department therefore should always be men of bridge engineering experience, capable of figuring bridge stresses and of doing first class bridge inspection. They should also be capable of getting up first class designs for new bridges needed at any point along their road—designs, such as can be given to bridge manufacturing companies for them to make intelligent bids upon.

Fig. 9, shows a design for a single track deck plate girder bridge for a 26 ft. opening. This particular bridge is designed in accordance with specifications of the L. S. & M. S. Ry. All necessary information is here given for all details of the bridge. The shear has been

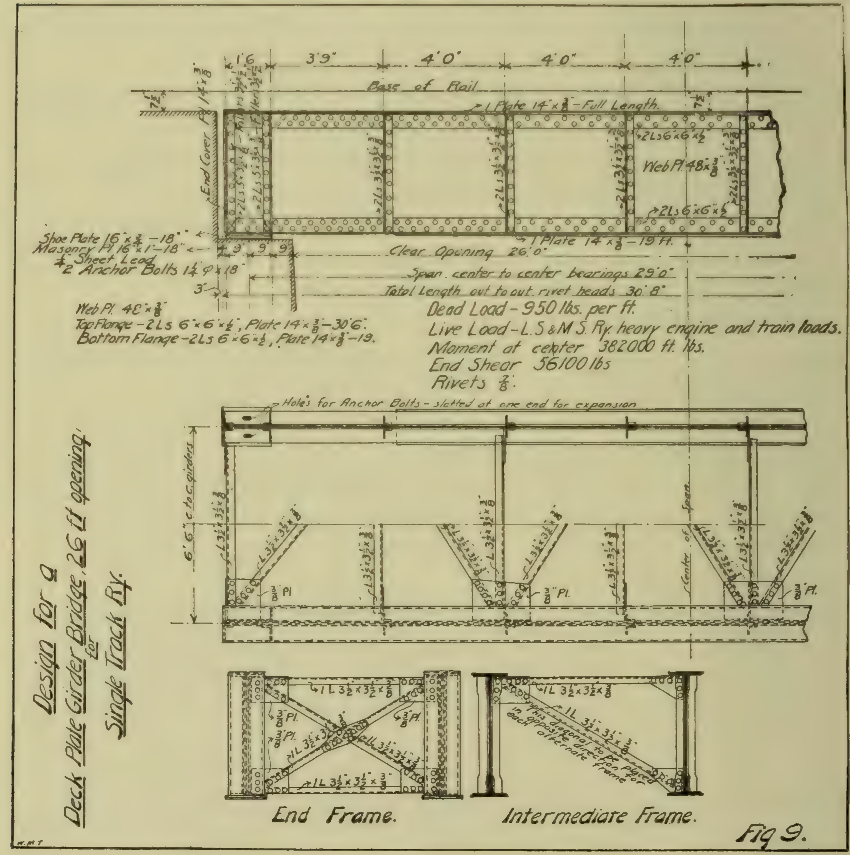

1'6
3'9"
4'0"
4'0"
4'0"
Base of Rail
1 Plate 14"×3/8"–Full Length.
7½"
End Cover Pl 14"×3/8"
2Ls 6"×6"×½
Web Pl 48"×3/8"
2Ls 3½"×3½"×3/8"
2Ls 5"×3½"×3/8"
1 Plate 14"×3/8"–19 ft.
Shoe Plate 16"×¾–18"
Masonry Pl 16"×1"–18"
¼" Sheet Lead
2 Anchor Bolts 1¼" φ×18"
9 9 9
3"
Clear Opening 26'0"
Span center to center bearings 29'0"
Total Length out to out rivet heads 30'8"
Web Pl. 48"×3/8"
Top Flange –2Ls 6"×6"×½", Plate 14"×3/8"–30'6".
Bottom Flange –2Ls 6"×6"×½, Plate 14"×3/8"–19.
Dead Load - 950 lbs. per ft.
Live Load - L. S & M S. Ry. heavy engine and train loads.
Moment at center 382000 ft. lbs.
End Shear 56100 lbs
Rivets 7/8".
Hole's for Anchor Bolts - slotted at one end for expansion
6'6" c. to c. girders
L 3½"×3½"×3/8"
3/8" Pl
Center of Span
Design for a Deck Plate Girder Bridge, 26 ft opening, for Single Track Ry.
1L 3½"×3½"×3/8"
3/8" Pl.
End Frame.
Intermediate Frame.
in this diagonal to be placed in opposite direction for each alternate frame
W.M.T.

Fig. 9.

figured at various points and the necessary rivet spacing in the flanges, to take care of the same, in addition to the weight of the train load above, has been figured and shown on the drawing.

Stiffener angles and lateral bracing, bearing plates, etc., have also been designed in accordance with the specifications referred to. The maximum end shear and maximum center moment have been computed for the loads taken and are shown on the drawing.

The sections for flanges and web of girder have been calculated from these quantities in accordance with these same specifications. The bridge shown above is a simple bridge in comparison to many which the maintenance engineer of any important railway will find necessary to design or to have designed for him.

It is highly important that the complete preliminary designs of all new bridges for the road, showing all stresses and sections desired, should be gotten up in the railway offices or in offices of their consulting engineers instead of in the offices of the bridge manufacturing companies. These companies would have too much interest in the matter of design, opposed to the interests of the railway company for them to make the best designs possible.

Much watchfulness is also required to obtain good jobs of erection on bridges contracted for by the railway company.

The work of erection will generally be done by the makers of the bridge and the erector in charge of the work will usually have no interest other than to get the work done as quickly and cheaply as possible. Unless carefully watched by the railway engineers and inspectors, he will be apt to slight the work in many vital points. To illustrate this point, a case which came under the writer's observation some months ago, will be cited. A draw bridge of some 230 ft. span was being erected for one of the large railway companies having a terminus in this city. The railway engineers in charge of the work were men of practical railway experience in many branches of the science but who had never had practical education in inspection of iron work. The bridge was nearly ready to be swung, the last pin connections about to be made, when it was found impossible to drive one of the pins. One of the posts connecting at that point could not be drawn into position, on account of projecting heads of rivets connecting the pin plates of the post to it. The erector, being in a hurry, had his men cut off these projecting rivet heads, thus destroying the rivets, instead of having them take out the pin at the other end of the post and turn the post 180° so that it would go to its position as designed. Had the work of inspecting the erection of this bridge been in the hands of competent inspectors, the erector would not have acted as he did, or else he would have been obliged to take out the post, redrive the rivets cut off, and replace the post in its proper position before the bridge would have been accepted.

It is a deplorable fact, however, that in the case of all but the best roads, it is the exception rather than the rule for bridge manufacturing companies to erect bridges for railway companies under proper field inspection. This fact often depreciates by a considerable amount, the value of the new construction obtained.

It is equally important to the railway company that all material which enters into the structure, whether steel, iron or timber, shall be of the quality desired. To insure this, careful inspection by experienced men, in the mills and shops where the steel or iron is made and assembled and also in the lumber yards should be made, and is always made under the supervision of the maintenance engineers of all first class roads.

The proper inspection of existing bridges is also an important part of a maintenance engineer's duties. This inspection should include the examination of material found in the bridges to determine how age has affected it and whether it needs to be renewed on that account. It should also include an examination of the stresses in the bridges, as many of them may be overstrained in many points because either of poor original design or the increased weight of locomotives and train loads in use on the road. If a bridge be found overstrained, no time should be lost in getting it renewed. The writer knew of the case of a large railway system, the policy of whose management seemed to be to keep up the bridges on the main track line at the expense of those on the numerous short side track lines belonging to that system. On a number of these latter were several timber bridges of some 50 ft. span each. The form of truss used in these bridges, designed some 10 years ago perhaps, was that of a half through Queen-post truss. The designer neglected, however, to put in proper diagonal bracing in the middle panel with the result that one of these bridges failed under a locomotive, doing much damage to the railway company not only in the destruction of the bridge and traffic across it for a considerable length of time, but in injury to the locomotive, its engineer and fireman. To the best of the writer's knowledge, however, the remaining bridges of this type on this railway system, are still in place and being used as they were previous to the accident spoken of. The division engineer in charge of that portion of the road is a business man and a mechanical engineer instead of a civil engineer which may explain that condition of affairs. There should be no excuse however, for such a flagrant violation of bridge design as that mentioned above.

LOCKS AND LOCK-GATES FOR SHIP CANALS.

BY HENRY GOLDMARK, A.B., M. AM. SOC. C. E.

The problems of canal construction as a part of the Civil Engineer's work have within recent years assumed new and unexpected prominence. Several important canals for a navigation of the first-class have lately been completed and further projects of unparalleled magnitude are now under construction or the subject of serious discussion.

Among the large works recently finished abroad may be mentioned the Manchester, the North Sea-Baltic, and the Corinth canals, and the enlargement of the canal prism at Suez and Amsterdam. In America the most important waterways under construction or survey are the great Drainage Canal at Chicago now nearly finished ; the rival projects at Panama and Nicaragua, and the equally important plan for a canal of the first-class connecting the Great Lakes with tidewater.

All this activity is the more striking, because for more than a generation the rapid development of railroads appeared to have given a death blow to new canal construction, and many existing canals had suffered a decrease in their traffic or been entirely abandoned.

There were, however, good reasons for this temporary decline which was not due to any inherent weakness in canals as such, but rather to a mistaken public policy by which their great advantages were not properly made use of. The superior economy of transportation by water with vessels of proper design and in waterways of considerably size is not open to question. The modern freight steamer, both on the high seas and our own Great Lakes, carries freight at a cost much less than even the lowest railroad rates. The tonnage of the lake traffic, particularly, has of late years advanced by leaps and bounds.

It is, perhaps, impossible to reach the same high degree of economy in the case of canal and river channels, which are necessarily more restricted. But in canals of large cross section, using modern vessels propelled by power, the cost per ton mile should not be much greater than in open water. The real reason why our canals have decreased so much in relative importance lies in the fact that in size, in construction, and especially in the nature of the boats used on them, they are many years behind the times and represent a phase of development long past in all other departments of transportation. When operated in competition with the highly developed railway systems, embodying the latest improvements of modern engineering, it is not to be wondered at that they have lost most of their former importance.

The only way in which canal navigation can be revived is to put it, as nearly as feasible, on the same footing as navigation in lakes and large rivers, by using large vessels, equipped with modern machinery, in channels of sufficient cross-section to keep the resistance to the movement of the vessels within economical limits. It goes without saying that canals of this description are very expensive to construct and maintain. There will, therefore, be but few locations on which the volume of the traffic will be sufficient to warrant their construction, and we may expect that but few canals will be built in the future, but they will be works of strictly the first class.

To the constructing engineer canal building offers many problems of great interest. The location of the canal, both from a commercial and a strictly engineering standpoint, requires careful study, while the excavation of the channel offers a field for introducing new and ingenious methods for handling earth and rock work on a large scale. The hydraulic questions involved, such as seepage, evaporation, problems of water supply, the flow of water in open channels, etc., are all interesting as matters of theory and offer a rich field for experimental research.

In this paper it is not proposed to take up any of these topics, but to confine it to the subject of canal *locks*, not only because they are the most important structures in canal construction, but also because they have not been adequately treated in American engineering text books.

GENERAL DEFINITIONS.

A canal lock may be defined as a structure which enables vessels to pass from a body of water to an adjacent one which is at a different level. As usually built, it consists of an enclosed basin or chamber provided with gates by which it may be shut off at either end so that it can be put in communication alternately with the upper and lower levels. The method by which boats are passed through a lock is simple, and readily understood. Besides the ordinary canal lock, various other means for overcoming differences of level in canals have been proposed at different times for at least 100 years past. Among these may be mentioned inclined planes and mechanical lifts acting vertically. A few inclined planes have been in use on small canals both in America and Europe for many years. Of vertical lifts a large number of projects have been worked out on paper, but only four of these have been built and are now in use. They are the hydraulic lifts at Anderton, England; Les Fontinettes in France, and La Louviere in Belgium, and the floating lift, so-called, at Henrichenburg in Prussia. The largest of these is the last named, which is 230 feet long, 28 feet wide, with a draft of water of 8 feet. The amount of lift is 52½ feet.

The operation of these lifts, the oldest of which has been in use for over 20 years, is quite satisfactory. Their principal *raison d'être* is the saving in water which they accomplish as compared with ordinary masonry locks. They are certainly of much interest, and in special locations their use will probably be more general in the future. The locks are, however, at best the most vulnerable portion of a canal system, and engineers may well hesitate before putting a more complex mechanism in place of the simple and massive masonry lock.

HISTORY.

The invention of the canal lock is one of the few great discoveries by which civilization has been measurably advanced. It alone has made it possible to navigate many important rivers and to carry canals over considerable elevations where a single level canal would be out of the question. The credit for building the first lock is claimed by both Holland and Italy, but the evidence as to time and place is conflicting. While in the plains of Northern Italy, the navigable canal is the outgrowth of the shallow irrigating ditches used from time immemorial, the Dutch canal for boats has developed from the channels required to drain the low lying fields or polders. In both countries simple sluice or head gates were built long before the enclosed lock with enclosed chambers. Such gates are sometimes used for navigation and are often confounded with true locks by the earlier writers. The first clear and distinct description of a lock with an enclosed chamber is said to have been given by Leona Battista Alberti in his book entitled "De re Aedificatoria," a copy of which he presented to the Pope Nicholas V. in 1452. Simon Stevinus, the celebrated Dutch scientist, also gives a good account of a canal lock in a treatise published in 1618.

By other writers it is claimed that the first lock was built in 1481 near Padua in Italy, while the advocates of Dutch priority feel confident that true canal locks were in use in the Netherlands before 1250. It may be added that the common canal lock is frequently called the *Visconti* lock from its alleged inventor, while by others the laurels of Leonardo da Vinci, already so ample, are increased by ascribing the discovery to the great painter. The exact date is, of course, not very important. It is of interest, however, to note that lock building as well as canal construction generally, antedates the establishment of our profession by several centuries. In hydraulic works of all kinds, many successive generations had accumulated a large amount of practical experience long before the civil engineer, as such, had come into being. In canal work, a high degree of perfection was reached at least 150 years ago. Faulty methods in construction and operation had been gradually eliminated by the severe test of time and experience so that the forms then in use have been followed pretty closely, at least for small canals, down to the present day. Within the past fifty years many large locks have been built, but the principles of their construction are essentially the same as those followed in the older and smaller works.

Although the ordinary canal lock has often been criticised on various grounds, it cannot be denied that it has proved itself in practice an extremely satisfactory piece of mechanism. It is simple and durable, requires few repairs and is inexpensive in operation.

CLASSES OF LOCKS.

According to their location, locks may be divided into two general classes:

(1) Locks in inland canals and canalized rivers.

(2) Locks in maritime canals and harbors.

In the first class, the difference of level to be overcome is due to the configuration of the ground which makes it necessary to divide up the waterway into a series of pools or reaches at different levels. The "lift" in this case is practically constant and the water pressure against the gates of the lock always acts in the same direction.

On the other hand, in the locks used in harbors and in canals communicating with the ocean, the difference of level is due to the tides and in certain cases to wind action. In the North Sea–Baltic Canal, for instance, there is a complete lock at the east end of the canal which is in use only about 25 days in the year, at times when a strong east wind from the Baltic piles up the water in the outer harbor.

The principal use of locks in harbors is for closing dock entrances where the range of the tides is considerable. This is the case on the coasts of England and Germany and on the Atlantic coast of France. The difference between high and low tide is rarely less than 15 feet, while in some localities, such as the ports in the Bristol Channel, it reaches 44 feet at certain times in the year. In these harbors vessels are loaded and unloaded in enclosed basins surrounded by quay walls in which the water is kept approximately at a constant level. These basins have narrow entrances, closed by one or more gates. In some of the docks, especially those of earlier construction, there is no enclosed chamber so that the lock reduces to a mere pair of gates in the entrance channel. These gates are open for an hour or so at high tide and all vessels must pass in and out at this time. When the tide in the outer harbor begins to fall, the gates are closed and keep the water in the dock basin from running out. In order to provide against exceptionally high tides in the outer harbor, another pair of gates is usually added which are built so as to support water pressure acting from the outside. A further modification where the range of tide is great is the introduction of a "half tide lock" with a second pair of gates so that the pressure on each of them is reduced.

The limited time to which the traffic is confined in this form of dock entrance is objectionable and many modern English docks are provided with complete locks having enclosed chambers so that vessels can be locked through between the outer harbor and the docks at all hours. At high tide the gates are left open for some time and the larger vessels usually come into the dock without locking.

The construction of these harbor locks is almost exactly identical with the locks on large ship canals. In the leading ports of Great Britain, a large number have been built in the last fifty years. In Liverpool alone there are more than 100 pairs of lock gates for openings varying from 40 to 100 feet. As but few large ship canals have so far been built, it is to the experience gained in building these large dock gates that we must look for guidance in designing similar works.

DIMENSIONS.

In designing a complete canal lock, the first points to be fixed are the proper dimensions. These are the length, the width, the depth of water on the sill and the lift or difference of level between the water above and below the lock.

The width and length, and the depth on sill are commonly the same for the whole canal and depend on the maximum size of vessel employed. On the canal proper, it is necessary to make the prism very much greater than the cross section of the vessel, say from 4 to 6 times as great, so as to reduce the resistance to motion through the water to an economical amount. In the locks this is unnecessary. An excessive size involves waste of water, increases the time required to operate the lock and greatly increases the first cost. In some cases where the traffic is very heavy, locks have been built wide enough to allow two ordinary vessels to be docked side by side and long enough to take in several of them, one behind the other. The new American lock at Sault Ste. Marie, which is 100 ft. wide and 800 ft. long, is a so-called "fleet lock" of this kind. The wisdom of this design is doubtful. As the width and length of lake vessels is constantly increasing, it will not be very long before all the older and smaller vessels will go out of service, so that the 100 ft. lock will not be wide enough to take in two of the vessels side by side nor long enough to allow them to enter "tandem." In that event the large dimensions of the lock will be worse than useless. The Canadian lock at the Sault, finished in 1895 is anly 60 ft. wide, but 900 ft. long and appears better adapted to the demands of traffic.

The probable size of vessels in the future is not easy to foresee and the dimensions to be adopted for designing locks for large ship canals will vary greatly according to individual judgment. Some thirty years ago the largest vessels were steamers with paddle wheels that projected a considerable distance on either side of the hull proper. To provide for these, several locks 100 ft. wide were built in the Liverpool and Havre docks. These are now far wider than necessary. At present few merchant vessels are wider than 60 ft., although a few of the largest exceed this limit and the Freidrich der Grosse is 68 ft. wide. War vessels have somewhat greater beam; the Iowa of the U. S. Navy is 76 ft. wide over all.

The locks on the North Sea-Baltic Canal are 82 ft. wide, while the new locks at Bremerhaven are to have a clear width of 92 ft. in accordance with the request of the North German Lloyd Steamship Company. On the Manchester Canal 80 foot and 65 foot locks are used although a narrower lock is built at the side for small craft. The proposed locks for the new Panama Canal are to be 59 and 82 ft. wide and about the same widths will probably be adopted at Nicaragua.

The depth of water on the sill of the lock, should equal the maximum draft of the boats with an additional clearance of 1½ to 2 feet. The "lift" of a lock is its most important feature. If the width may be compared to the "length of span" in a bridge, the lift is analogous to the loading to which the bridge is subjected. The lift or difference of level is fixed by topographical configurations, though in many cases the location of the canal is affected by the amount of lift which can safely be used. The inferior limit of the lift, in a lock may be 1 ft. or even less. The upper limit has not yet been reached. Very few locks with lifts exceeding 20 to 25 feet have ever been built. The greatest lift known to the writer in an inland canal lock is 30 ft. This lift is used at the new locks in St. Denis Canal in France. In the Avonmouth dock at Bristol, England, the range of the tide is nearly 44 ft. and the strength of the gates is calculated for a head of 45 ft. This lock was built nearly thirty years ago and though the gates are of timber, their operation has been entirely successful.

The question whether lifts as high as 40 or 50 feet are advisable must be studied carefully for each separate case and will depend on the supply of water, the density of traffic and other considerations as well as on the structural difficulties involved. During the past year the writer has been engaged in the design of locks of various lifts up to 50 feet. So far as his plans have been matured, they show no reason why lifts of 45 or 50 feet could not be successfully used on locks as wide as 80 feet.

Such great lifts will seldom be needed as the topography of the country passed through is almost always such as to make the majority of locks of moderate lift. Even where a concentration of the locks at a few points might otherwise be advantageous, this can rarely be done without flooding too large an area of valuable land. For this reason the opinion sometimes expressed that the adoption of mechanical locks which permit the concentration of the lift at a few points will always result in economy, is a mistaken one.

CONSTRUCTION OF LOCK WALLS.

The construction of a lock may be divided into three parts.

(1) The foundation, the side walls and the floor which are generally built of masonry.

(2) The culverts and valves for filling and emptying the lock, with the mechanism for operating the valves.

(3) The lock-gates and the machinery for moving them.

As in most structures, the nature of the foundation encountered affects the difficulty of construction to a high degree. Fortunately in inland canals, the locks can often be located on a solid rock bottom. In the case of harbors, on the other hand, rock is rarely encountered and in many cases the bottom is extremely soft. The successful operation of the gates requires that the side-walls and sill should remain almost absolutely true to their original lines. The difficulty of securing this result is very much greater than that encountered in building an ordinary quay wall. No general directions can be given as to the best choice of foundation in any given case. When the bottom consists of a rather firm sand or clay, it is usual to cover the entire site with a layer of concrete of sufficient thickness to support the upward thrust of the water which may tend to lift it. This layer of concrete is laid in the dry when this is feasible, but must usually be deposited under water. The side walls are built on this foundation and the portion between the walls forms the floor of the lock. When the bottom is softer and more variable, piling must be resorted to, at least under the side walls, so that the weight of the walls may not tend to crack the floor. The problem of dimensioning the side walls and the floor when the bottom is soft is extremely complicated.

When built on solid rock, a lock wall can be designed according to well understood rules in the same way as a retaining wall. Each wall acts separately and its weight is carried by the rock bottom immediately below it. The forces tending to overthrow the wall are the earth pressure behind it, to which must be added a certain amount of water pressure varying with the permeability of the back filling. In this calculation the lock is, of course, supposed to be empty and the ground water to stand at its highest level.

When designing a lock to be built on a soft bottom, we cannot calculate the strength of each wall separately but must consider the entire cross section of the lock, *i. e.*, the two side walls and the concrete floor as a whole. This section is subjected to a variety of forces, viz.: the earth and water pressure on the side walls, the upward pressure on the bottom of the floor and the walls besides the weight of the masonry and of the water in the lock. The magnitude and distribution of the upward reaction of the bottom cannot be exactly estimated. It is possible, however, to make a graphic analysis and draw a line of pressures in the walls and floor under various hypotheses. By comparing the conclusions to be drawn from this analysis with practical experience in locks built on a soft bottom, much assistance can be gained in proportioning new structures. With the usual proportions the line of pressure at the middle of the floor is quite eccentric. This shows the existence of a considerable bending moment which would tend to crack the floor at the top. Such longitudinal cracks have actually occurred in a number of harbor locks at the very points indicated by the theoretical analysis. They are not necessarily of a destructive character and after they have been closed with concrete are not likely to give much further trouble. The structure after fracture is in a new position of equilibrium corresponding to a new distribution of pressures on the bottom.

Laying concrete under water is always somewhat unsatisfactory. In building the large locks at Holtenau, on the North Sea-Baltic Canal, a simple but elegant method was used for lowering the ground water level and excluding the water from the lock pit. Three wells, 12 ft. in diameter, were sunk by compressed air to a depth of about 15 feet below the bottom of the pit. They were placed close to and just outside the lock at three of its corners. By pumping from these wells with centrifugal pumps for a period of 15 months, the water level over the entire lock was lowered so that the foundation could be built entirely in the dry.

Compressed air caissons and open wells sunk by dredging have also been used for the foundations of harbor locks. The method used is practically the same as that employed for bridge piers. The locks at Toulon, Dieppe and some other French ports were built with compressed air foundations, while the Bordeaux lock was founded on open wells. In the latter case, the close proximity of large warehouses was the reason for chosing this method.

The material used in lock walls is almost always masonry, but floors of timber construction are not unusual even in large locks. Cut stone masonry is generally employed, though rubble with an ashlar facing is not uncommon. Of late years some locks have been built entirely of concrete. Among these are the fine locks recently completed by the U. S. Government on the Hennepin Canal in Illinois. The writer has also had occasion to examine the masonry recently built for the new guard gates in the St. Mary's Falls canal. This masonry consists of a rich concrete without any cut stone and presents a very good appearance. The gates are of timber and span a clear opening of 108 ft. This is a greater width than any known to the writer elsewhere.

The masonry at the ends of a lock must support the pressure of the gates. The walls at the ends are necessarily thicker than the side walls of the chamber and must be built with extreme exactness so as to fit the gates. Their details will depend on the style of gate used.

The construction of the masonry is further complicated by the necessity of inserting culverts for the filling and emptying of the lock and also of tunnels for the cables that move the gates and the pressure pipes connected with the operating machinery.

ARRANGEMENTS FOR FILLING AND EMPTYING THE LOCK.

Three different plans are in use for this purpose.

(1) Valves in the upper gate.

(2) Side Culverts in the lock walls.

(3) Culverts under the floor of the locks.

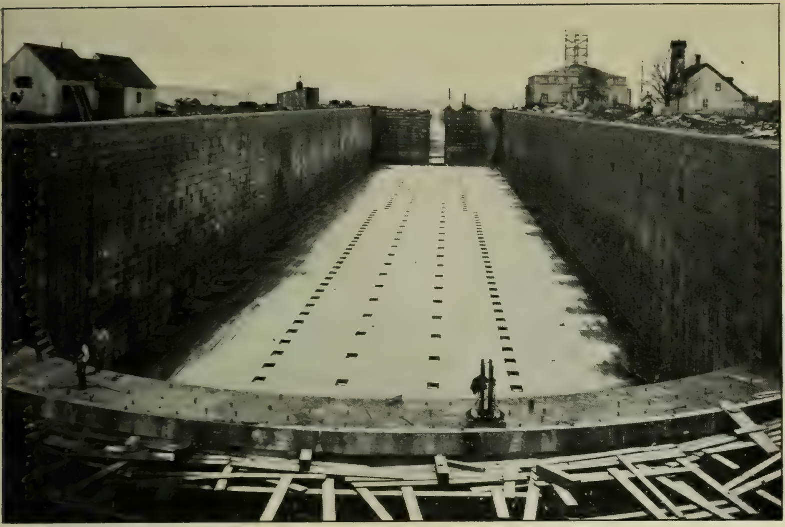
The first plan has the merit of simplicity and is generally used in small locks. The openings are rectangular and placed as low as possible in the gates so as to act with the largest possible head. The valves are simple sluice gates operated by hand from the top of the gate. Such openings weaken the gate where the water pressure is greatest. Another objection is the fact that the water rushes in with much velocity and tends to break the cables of vessels in the lock. Furthermore the time required to fill a large lock by valves in the gates is excessive. For this reason, such valves are supplemented or replaced in most large locks by culverts in the side walls or under the floor. The latter arrangement can be conveniently adopted only in case of a rock foundation, to which the floor system can be bolted down to resist the upward pressure of the water, tending to lift the floor when the culverts are filled. The most important examples of such culverts are found in the three great locks at Sault Ste. Marie. In all of these the water is admitted through large rectangular culverts under the floor. They are about 8 ft. square and connect with the lock chamber by a large number of openings along the bottom of the lock. The culverts run side by side and are built of solid timbers. There are two culverts in the smaller American lock, six in the larger and four in the Canadian lock. The head is about 19 feet. The largest lock is filled in about 11 minutes, using four culverts only.

Side culverts are general in the larger European locks, such as those in the Manchester and North Sea-Baltic canals. There is a culvert in each wall about twice as high as it is wide. In the Manchester Canal the size of the culverts is 6 × 12 feet. They discharge into the lock by lateral openings.

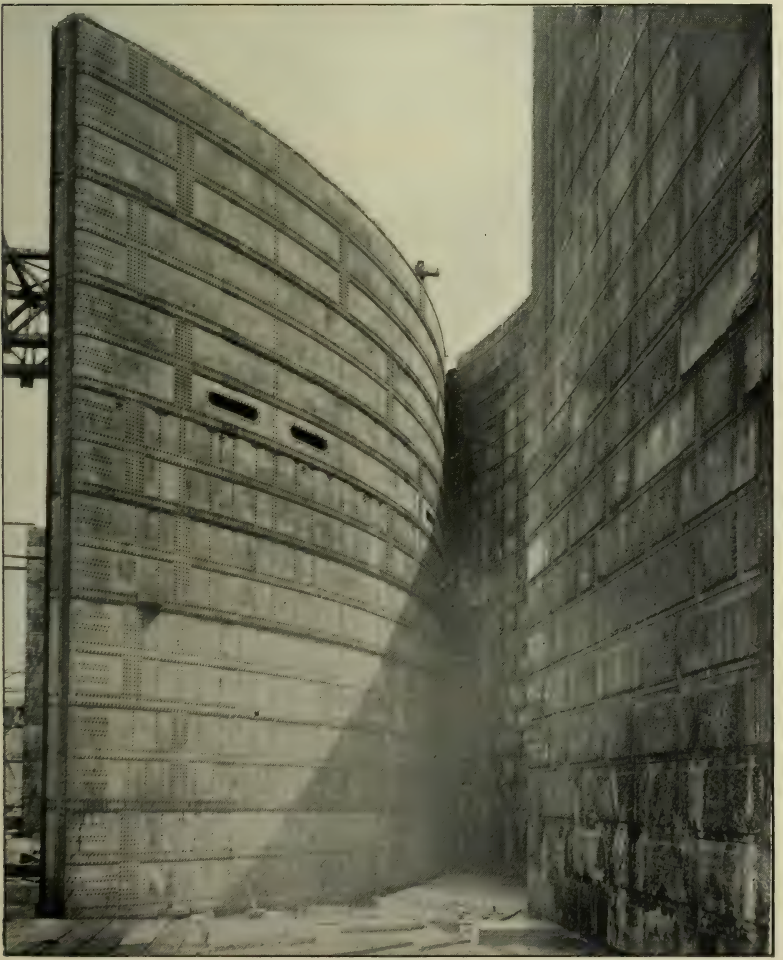
In connection with culverts, three classes of valves are used, viz.: Slide valves, butterfly valves and cylindrical valves. The first class are rectangular and may be built of either metal or wood. It is desirable that they should move with little friction and be as nearly water tight as possible. On the Manchester Canal the Stoney sluice gates are very successfully used, in which the friction is largely reduced by a system of roller bearings. In the North Sea-Baltic Canal a similar sliding gate built of timber was adopted. In American locks butterfly valves revolving on a central axis are common. They are simple in design and durable and require but little power to operate them. The only objection to their use is the excessive consumption of water, as they cannot be made with a tight fit. This precludes their use where water is scarce.

Cylindrical valves are in use on many French canals and have been

ST. MARY'S FALLS CANAL.



800 FOOT LOCK—ST. MARY'S FALLS CANAL.



SOUTH LEAF—INTERMEDIATE GATE. 800 FOOT LOCK.

proposed for the enlarged Erie Canal. They consist of vertical steel cylinders resting on a conical seat, and are raised vertically to admit the water through an annular orifice. While these valves have many good features they are quite expensive as the amount of water that can pass through any one valve is comparatively small. Valves are generally operated by power, the machinery being combined with that for moving the gates.

LOCK–GATES.

Although they represent a relatively small part of the total cost, the gates are more complex in construction than any other part of the lock, and on their correct design its successful operation will largely depend. Considered merely as structures, they present an interesting field in the theory of stresses and in practical designing.

Every lock with an enclosed chamber must have at least two gates, one at each end. Besides this an intermediate gate is frequently added which permits the working length of the lock to be shortened so that smaller vessels can be locked through more quickly and with less waste of water. Quite generally, too, a guard gate is built at either end to allow the entire lock to be laid dry for periodic examination and repair.

Lock gates, whatever their detailed design, are really movable dams, and when closed support the pressure of a considerable head of water. The standard form used in the great majority of cases is the mitering gate. This consists of two leaves, each turning on a vertical axis like an ordinary door. When closed the leaves meet at an obtuse angle, the so-called toe-posts abutting against each other in the middle of the lock, while the bottom of the gate rests against a continuous sill. When in this position the two leaves act as an arch, which conveys the water pressure to the side walls. The fitting of the gates against each other and the sill is difficult to make and maintain uniform at all times. A bad fitting may interfere with the proper working of the gates, and also causes the stresses in the different members to be somewhat uncertain.

For these reasons among others, many substitutes for mitering gates have been proposed and some of them carried into execution. The more important of these forms may be briefly referred to.

(1) The single leaf revolving gate. This consists practically of one leaf of a mitering gate long enough to reach across the lock at right angles; the gate is supported on the bottom and both sides and acts as a girder or truss instead of an arch. The single leaf is, of course, heavier than the separate leaves of a mitering gate for the same open-

ing. It requires much more power to move it and also shortens the available length of the lock which can be occupied by vessels. The cost is about the same as for double leafed gates. Single leafed gates have of recent years been built in France up to 50 feet in length.

(2) The "Tumble" gate which also spans the canal with a single leaf, but revolves on a horizontal shaft fixed at the bottom of the lock. This form has been used in some of the Erie Canal locks for many years.

(3) Sliding Gates. Gates of this kind have been built in different English and continental harbors and in this country in connection with the Davis Island Dam in the Ohio river improvement. The foreign gates are of iron with closed air chambers, while the Davis Island gate which spans an opening of 110 feet is of timber framing. These sliding gates when closed, act as trusses supported by the side walls and the sill. They are opened by moving them sideways at right angles to the lock into a recess constructed in the masonry wall on one side.

(4) Pontoons. Pontoons are sometimes rectangular gates like the sliding gates just referred to, although they may also be built having the curved outlines of an ocean vessel. They are floated across the lock entrance and are sunk into position by letting water into tanks provided for the purpose. When the lock is opened they are moved into recesses in the wall. Pontoons are used generally in dry docks, but are not well adapted for ordinary canals where rapid and frequent moving of the gates is required. The same may be said of the sliding gates although the latter if properly designed and fitted with a good moving mechanism would probably give good satisfaction in canal work.

The ordinary mitering gate has, however, in the writer's opinion, so many strong points in likeness, durability and facility of movement that it is likely to hold its own even for large locks.

MITERING GATES.

Mitering gates are built of all sizes from the great gates spanning an opening of 100 feet down to the smallest guard gates. The material used in their construction is timber or iron or a combination of the two. For small gates, timber is in every way preferable as the first cost is less, repairs are more easily made and there is no difficulty in designing gates of simple construction using timbers of small scantling and length. A number of small iron gates have been built in different countries, but the concensus of opinion among the Engineers directly in charge of canals seems to be averse to their general adoption.

The general use of steel in bridges and ships, makes large wooden lock gates seem somewhat out of date. Metal would appear to have great advantages as in other engineering structures. Large iron gates, have as a matter of fact, been in use for over fifty years, the first wrought iron gate having been built for a 60 ft. dry dock entrance in the Brooklyn Navy Yard about 1850, while about the same time similar gates were constructed by English engineers at Sebastopol, Russia. It has never been denied that these and later iron gates have given perfect satisfaction.

It is true, nevertheless, that many English and American engineers of great experience in lock work remain strongly prepossessed in favor of timber gates. In England even at the present day, about half of the new gates are built of wood. In the Manchester Canal Greenheart timber, a very durable wood, brought from British Guiana was used exclusively in the 54 gate leaves built, although the cost was much greater than that of iron gates. Some of the larger American gates such as those in the new Canadian lock at the Sault are also built of wood.

Apart from natural conservatism, the reasons which make for wooden gates are, their greater lightness which makes them easier to move and still more the ease with which they may be repaired in case of a collision. Such accidents are always possible, although they are rare. It does not seem to the writer, that this contingency is sufficiently probable to make it wise for us to give up the great advantages of steel gates.

DETAILS OF CONSTRUCTION.

A mitering gate consists of a skeleton or *frame* and a *water tight* sheathing. The frame may be arranged in different ways, but there is always a heel or quoin post close to the masonry, a toe or mitre post at the other end of the leaf and two horizontal girders, one at the top and another at the bottom of the gate. Besides this there are usually a number of intermediate horizontal girders forming a series of arches or rafters carrying water pressure. In a few gates, vertical girders which bear against the top horizontal girder and the bottom sill take the place of the intermediate horizontals. The weight of the gate is supported on a vertical pivot fastened to the bottom of the quoin post, while at the upper end of this post, there is an anchorage which extends into the masonry wall. A roller traveling on a circular track on the bottom of the lock has in the past been quite generally used at the outer end of large gate leaves. This relieves the pivot and anchorage of much weight but makes distribution very uncertain. The disadvantages of rollers have led to their gradual abandonment.

TIMBER GATES.

The sheathing is always made of planking with calked joints. The posts consist of one large timber or are built up of several pieces. The horizontals differ in construction according to the size of the gate. For moderate spans straight horizontals made of a single timber can be used, but for larger gates built-up trusses must be employed. Where long timbers can be had bowstring girders with iron tie-rods or tie-beams of wood are probably the best forms to be adopted. As examples of such girders, the old gates for the 100-foot dock entrances at Liverpool and Havre and the Weitzel lock at the Sault (60 feet wide) may be referred to. The gates in this last lock have been renewed during the past winter. They were designed by Mr. Alfred Noble, M. Am. Soc. C. E., and completed under his care in 1881. The iron rods, pivots, etc., were found to be in perfect condition and have been used for the new gates.

When long timber is difficult to obtain, the horizontal girders may be built up of several short lengths framed between vertical intermediate posts and bolted to re-inforcing timbers. Many English gates are built in this way.

IRON AND STEEL GATES.

Iron or steel gates like timber gates consist of a frame and a sheathing, both of metal. The cushions at the quoin and mitre posts and the sill where a water-tight closure is required are usually made of wood.

The design of a steel lock-gate, like that of any other structure, is largely dependent on the forces which it has to resist. These will be different when the gate is opened and when it is closed. When open the gate exerts a horizontal pull on the anchorage while its weight rests on the pivot. These forces must be transmitted through the gate frame and are readily analyzed.

When the gate is closed the water exerts a horizontal pressure, which is transferred by the gate to the side walls and sill of the lock. The magnitude of this pressure is easily determined, being at each point equal to the hydrostatic head. The upper gate is most strained when the lock is entirely empty. The pressure increases from zero at the top to a maximum at the bottom and may be represented by a triangle.

In the lower gate it is zero at the top increasing uniformly to the level of the lower pool and from that point is a constant to the bottom of the gate. It may be represented by a trapezoid.

The gate can be designed to stand this pressure in various ways. The most common form consists of a series of horizontal girders spaced in an approximately equal manner and fastened securely to the quoin and mitre posts. They are further held in place by vertical frames, intermediate between these posts, which add greatly to the stiffness of the gate. The sheathing consists of plates riveted to the horizontals and calked at all joints to secure water-tightness. This sheathing is required only on one side as far the function of the gate as a dam is concerned. It is a very general practice, however, to place the covering on both sides, forming a series of air-tight compartments, the flotation of which relieves the pivot and anchorage of weight and makes the gate easier to turn. Some of the chambers are also filled with water as ballast.

The closed chambers are hard to keep tight and somewhat inaccessible. For this reason in some of the latest designs, such as the Cascade Locks on the Columbia River and the Plaquemine Locks in Louisiana, both built by the U. S. Government, they have been omitted and the gates built with a single skin only.

In the beginning the actual design, the first point to be settled is the rise of the sill which fixes the angle which the gates make with the axis of the lock. The rise varies from $\frac{1}{3}$ to $\frac{1}{6}$ of the width in various locks, but a rise of $\frac{1}{5}$ is perhaps the best, being as economical as a greater rise.

The next point to be considered is the proper outline of the horizontals. These are almost always plate girders. They may have a girder shape or else follow the lines of an arch the medial line of which is a circular curve through the centre of the quoin and mitre posts.

Each horizontal is in equilibrium under three external forces, viz. : the water pressure, which is uniform and normal to the face of the gate, the reaction of the other leaf which is at right angles to the axis of the lock and the reaction of the masonry at the quoin. These two reactions are equal and make the same angle with a line connecting the centre of pressure at the quoin and mitre posts.

If the gate consisted of a linear arch without thickness, a circle would be the true line of equilibrium for the forces acting on it and the arch would be in pure compression and hence the most economical shape. On these theoretical grounds, it has generally been held that an arch gate of circular shape is necessarily the most economical. This has been stated by many different writers for 50 years back and the proposition has been re-inforced by many intricate calculations involving the use of the higher mathematics.

As a matter of fact, the gates are never linear arches but must be built as curved beams which are rarely less than 3 feet in thickness so that the surface submitted to the water pressure is not identical with the curved axis of the horizontal girder. Furthermore the center of contact or pressure where one leaf presses against the other at the mitre post is rarely exactly on the medial line, but on the contrary varies considerably on either side. This difference of position is due both to unavoidable inaccuracy in fitting and material and also to the change in the length of the gate leaves at different times owing to variations in temperature. As a result of this the circular arch is never in pure compression but also subject to considerable cross-bending. Besides this in proportioning engineering structures many practical considerations, such as the minimum thickness of metal, etc., that may be used must be taken into account, so that any general theoretical deduction loses still more in value.

The only reliable method of comparison for different shapes consists in a series of estimates based on actual detailed designs. By means of several extended estimates of this kind the writer has satisfied himself that at least for locks up to 80 feet in width, the circular arched gate is no more economical than the straight or girder shape, while it has many practical disadvantages.

The dimensions of the web and flanges in any given girder are to be determined by the rules commonly used where there is a combination of compressive and bending stresses.

Another interesting question is the distribution of the total water pressure over the different horizontal girders. The total amount of this pressure for the whole gate is perfectly determinate. In case the horizontal girders were connected by a flexible sheathing, the distribution would be equally simple, each girder getting exactly the load due to its head below water level. As actually built the girrders are connected by sheathing that has some stiffness and by vertical posts that have much rigidity. Furthermore, the bottom of the gate fits more or less closely against a solid sill. The stiff vertical members modify the distribution of the load over the different horizontals, even when there is no contact on the bottom sill and still more when there is contact so that the verticals carry some of the water pressure to the bottom sill. The result is that the upper part of the gate is more fully loaded, while the lower horizontals are proportionately relieved.

Some interesting experiments on models made by M. Chevalier in France in 1850, illustrate this point very beautifully. The mathematical statement of these complex stresses has also been attempted by several French engineers, but their methods are extremely intricate and the results while indicating correct values, hardly merit extreme confidence.

The method of "Least Work" for solving indeterminate stresses has been applied by the writer to this question with results that agree satisfactorily with some measurements he has made during the past year in the deflections of large gates.

French Engineers commonly design the lower girders of their gates in accordance with the formulæ referred to above, assuming simultaneous contact of the mitre post and sill at all times, while in England it is usual to proportion each girder for its full hydrostatic head. As the close fitting at the sill is likely to fail at times, the English practice seems the safer one, though the upper part of the gate should be strengthened rather more than is customary in some of the English gates.

The details of construction in all parts of the gate will, of course, vary according to the individual judgment of the engineer in charge.

Many otherwise good gates are unnecessarily complex in construction, showing a lack of familiarity with shop practice on the part of their designers.

In lock-gates, which are machines rather than structures, facility of operation and freedom from breakdowns are far more important than first cost. At the same time, a gate that is simple in detail is also likely to be satisfactory in daily use.

MACHINERY FOR OPENING AND SHUTTING THE GATES.

The methods used for opening and shutting the gates can only be briefly referred to. In large modern locks the machinery is always operated by power in order to shorten the time required. The prime movers are generally turbine wheels operated by the water in the canal at a head equal to the lift of the lock. The power thus generated is transmitted to the mechanism for moving the gate by water under pressure, by compressed air or by electricity. In the past, water under pressures varying from 100 to 800 lbs. has been generally used. Machinery of this kind was first designed by Sir William Armstrong for English harbor locks, and includes the use of his well-known accumulator. Most English plants have been constructed by this firm and designers in other countries have generally used very similar forms. The water under pressure moves reciprocating pistons to which the cables are attached, or else rotary engines, usually with three cylinders, are used.

The turning of the gates is generally effected by steel cables or chains which are attached close to the mitre post near the bottom of the gate. One cable serves for opening and another for closing. The cables are brought to the engines on the top of the lock walls, through tunnels built in the masonry. The details of the attachment and general arrangement differ in various designs, but it is usual to have an independent engine on each side wall.

Although cables and chains have worked very satisfactorily, they have some disadvantages and in several recent locks other appliances for opening and shutting the gates have been adopted, thus in the new locks at Barry, in England, a stiff strut is used which is attached to the gate above the surface of the water and serves both to open and shut it. One end of this strut connects directly to a plunger that moves in an hydraulic cylinder. This cylinder oscillates on a double axis which is placed in a recess built in the wall approximately at right angles to the face of the wall. In the North Sea-Baltic Canal and also at the west end of the Amsterdam Canal a similar arrangement is used, but the strut is not directly moved by hydraulic power but carries a rack that connects with geared spur wheels.

Quite recently electric motors have been substituted for water pressure engines and the use of this power is likely to become general.

Hydraulic machinery in cold climates is always likely to give trouble and in some instances it is necessary to use oil in place of water during the Spring and Fall before it becomes necessary to cease operations entirely. Besides this, the transmission of power by pressure pipes to distant parts of the large lock involves expensive construction and repairs are frequently needed. The use of the electric current would seem to obviate all these difficulties. In the Canadian lock at Sault Ste. Marie, electric motors are used for opening and shutting the gates as well as operating the large valves in the culverts. The operation of this machinery is entirely satisfactory, although it seems to be rather complicated.

Electric power has also been adopted for the gates of the new lock at Ymuiden on the Amsterdam Ship Canal, as a result of an extended series of experiments. We may expect that in the future most of the new locks will be operated as well as lighted by electricity.

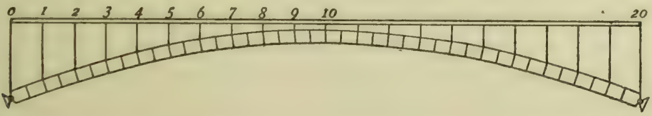
ON THE DESIGN OF A TWO-HINGED ARCH RIB.

HENRY S. JACOBY, ASSOCIATE PROFESSOR OF BRIDGE ENGINEERING AND GRAPHICS.

The graduation thesis of Mr. Harley S. Gibbs, of the class of '98, of the College of Civil Engineering, consisted of an investigation of the relative influence upon the design of a two-hinged parabolic arch rib of the moment of inertia of its cross section, and of the shortening of the rib due to the axial thrust or compression. The arch rib which was chosen for this purpose is the one used as an example in Arts. 77–83 of Merriman and Jacoby's Text-Book on Roofs and Bridges, Part IV. This was done to enable the investigation to be carried further in the limited time available than would otherwise have been possible. The span is 258 feet, and the rise 26 feet. The weight of the horizontal roadway and its loads is transmitted by equidistant vertical columns to the ribs which are thereby divided into twenty parts. The dead and live panel loads are 59.0 and 18.2 kips respectively, a kip denoting one thousand pounds. Nine designs were made by Mr. Gibbs, six relating to this arch rib under different conditions, and three of another rib of double the rise so as to show how the ratio of rise to span modifies the effect of the other conditions.

THE SIX CONDITIONS.

1. In the first design the moment of inertia I is assumed to vary as the secant of the angle of inclination i of the axis of the rib. This condition is expressed by the formula $I = I_c \sec i$, in which I_c is the moment of inertia of the rib section at the crown. The bending moments, axial thrusts, and shears were determined under each panel point, section *o* being at the hinge and section *10* at the crown. The methods of finding the stresses and designing the flanges and web are those given in the text-book referred to above, the effect of the rib shortening in Art. 80, being, however, omitted. The sections were re-



vised until the assumed value of I_c agreed with the final value. It was found that the maximum flange stresses were due to negative moments at all the sections except number *8*. The results for only the alternate sections are recorded in tables I and II. The flanges are designed to take the bending moments and axial compression, the web taking only the normal shear in accordance with the usual specifications.

2. In the second design the moment of inertia is assumed to vary as in the preceding case, the effect of the rib shortening being considered. The value of the horizontal thrust H at the supports was computed from formula 74 in Chap. II of Howe's Treatise on Arches. This formula is derived from an analysis which includes the effect of rib shortening leaving as the only approximation in the design that which is due to the moment of inertia. The value of the square of the radius of gyration substituted in the formula was the average value 6.38 feet^2, obtained in the preceding design. The values were found to range from 6.371 to 6.393, the average being 6.388 feet^2. No revision was made. The reduction in the value of H due to rib shortening in turn modifies the reaction locus. The only effect on the live loading, however, is to change it from panel loads *1–9* to *1–10* inclusive for the greatest positive moment in section *7*, and from panel loads *6–11* to *5–11* inclusive for section *9*. A corresponding change is made in the loading for the greatest negative moments in these sections. It may be added that this change in loading applies to all the designs in which the rib shortening is taken into account. The maximum flange stresses were found to be due to positive bending moments at all the sections except number *1* where they are due to a negative moment, and at section *o* where there is no bending moment and where the axial compression is a maximum under full load. The values of H are only 1¾ per cent. less than those in the first design, while in the table in Appendix C of Howe's Treatise the difference is given as about 7 per cent. This seems to indicate the unreliability of that table for spans differing materially from that for which it was computed, viz., 100 feet. The formula for H shows that its value does not vary simply as the ratio of the rise to the span. Especial attention is called to the fact that the differences between the corresponding values of H do not serve as a measure of the differences in the flange sections required, for in this design the flange areas are over 4 per cent. greater than those in the first design as shown in Table II.

3. In this case the moment of inertia is assumed to be constant, its magnitude being the average value obtained in the first design. The computations were made by means of the summation formulas 242 and

246 in Chap. II of Howe's Treatise on Arches, omitting the terms containing F_x, the sectional area of the flanges, since the effect of the rib shortening is here neglected. The method of using these formulas is explained in Chap. VIII. The co-ordinates x and y were taken to the center of each $\triangle s$. If they be taken to the ends instead of to the centers of these divisions the values of H are found to be 0.3061, 1.3728 and 1.9835 instead of 0.3105, 1.3848 and 1.9389 for a load unity at sections *1*, *5* and *10* respectively.

4. The fourth design differs from the preceding one by considering the effect of the rib shortening. The summation formulas 242 and 246 were employed substituting in them the average values of the moment of inertia and of the area of the flanges which were obtained in the second design. On using the modified formula as recommended by Howe in Chap. VIII, the approximate values of H were computed and found to be 0.3039, 1.3598 and 1.9040 instead of 0.3047, 1.3628 and 1.9080 for a load unity at sections *1*, *5* and *10* respectively. The difference increases with the rise of the arch and accordingly the formula for H was used without any modification except to re-arrange it slightly for convenience in computation.

5. The fifth case differs from the third only in using different values of the moment of inertia. It was assumed that the values for sections *0* to *4* would probably be those obtained in the third design, while those for sections *4* to *10* would be those of the first design. The composition of the flanges proved, however, to be exactly the same throughout for the third and fifth designs. No revision was made as previous computations indicated that the change would be very small.

6. In this design the true value of the moment of inertia is used at each section, and the effect of the rib shortening is considered. The same formulas are employed as in the fourth design, the values of the moment of inertia and of the area of the flanges for that design being substituted. On completing the design it was found that no revision was required since the results agreed with the values assumed. The numbers of the preceding designs are entered in the headings of Tables I and II.

DISCUSSION OF THE RESULTS.

The following results are shown by a comparison of the required flange areas in Table II :—

First, that the effect of the rib shortening due to axial compression should always be considered when the arch has a rise of about one-tenth of the span. If the flange areas for the sixth design be taken as 100 per cent., those of the first design, when $I = I_c \sec i$, are seen to vary from 89.9 to 104.3 per cent. ; those of the third design, when *I* is constant, range from 91.8 to 105.3 per cent., and those of the fifth design vary from 91.5 to 105.3 per cent. ; the minimum and maximum percentages in each case being those of sections *10* and *2* respectively. These differences from the true values exceed the allowable limit.

TABLE I.

	Section.	Rib Shortening Considered.			Rib Shortening Neglected.		
		$I=I_c$ sec. i. (2)	Ave'ge I (4)	True I (6)	$I=I_c$ sec. i. (1)	Ave'ge I (3)	True I (5)
Horizontal thrust H for a load unity placed successively at the sections given.	2	0.5978	0.6020	0.6039	0.6084	0.6117	0.6088
	4	1.1310	1.1370	1.1366	1.1511	1.1553	1.1504
	6	1.5439	1.5540	1.5517	1.5759	1.5791	1.5767
	8	1.8137	1.8179	1.8168	1.8457	1.8472	1.8526
	10	1.9044	1.9080	1.9085	1.9381	1.9389	1.9492
	1–19	24.320	24.412	24.408	24.759	24.807	24.808
Dead load bending moments. (Kip-feet.)	2	+269.8	+218.6	+220.8	+27.0	+0.5	−0.1
	4	+478.7	+388.6	+392.6	+47.9	+0.8	−0.2
	6	+628.6	+510.0	+515.2	+62.9	+1.1	−0.2
	8	+718.5	+582.8	+588.7	+71.9	+1.2	−0.2
	10	+748.5	+607.2	613.3	+74.9	+1.3	−0.3
Dead load axial thrust. (Kips.)	0	1538.8	1544.4	1546.6	1564.9	1567.9	1569.3
	2	1518.7	1524.0	1525.8	1545.3	1548.0	1546.4
	4	1483.8	1489.9	1490.6	1510.8	1513.3	1512.2
	6	1457.9	1463.8	1464.7	1484.8	1487.5	1486.3
	8	1440.0	1447.2	1448.0	1467.7	1470.2	1470.5
	10	1434.9	1440.3	1440.1	1460.7	1463.7	1463.6
Greatest live load positive bending moment. (Kip-feet.)	2	1115.2	1108.7	1109.1	1091.5	1088.1	1091.3
	4	1589.5	1577.1	1578.0	1538.2	1528.2	1535.3
	6	1531.9	1513.9	1515.2	1451.9	1442.0	1443.7
	8	1087.1	1064.8	1066.2	980.4	968.5	965.8
	10	797.9	785.6	787.9	693.8	693.3	668.7
Corresponding axial thrust due to the same loading. (Kips.)	0	474.7	476.4	477.1	482.7	483.7	484.1
	2	155.8	156.4	156.9	158.2	158.9	158.2
	4	178.6	179.3	179.6	181.7	182.3	181.8
	6	206.4	207.1	207.4	210.0	210.7	210.3
	8	237.7	238.9	239.3	242.3	242.9	243.1
	10	231.2	231.7	231.6	235.2	235.4	236.2
Greatest live load negative bending moment. (Kip-feet.)	2	1032.0	1041.3	1041.0	1080.7	1087.9	1091.3
	4	1441.8	1457.2	1456.9	1523.6	1528.0	1535.4
	6	1338.0	1356.6	1356.3	1432.0	1441.7	1443.8
	8	865.5	885.0	884.6	957.4	968.1	965.9
	10	567.0	598.3	598.7	672.3	692.9	668.8
Corresponding axial thrust due to the same loading. (Kips.)	0	474.7	476.4	477.1	482.7	483.7	484.1
	2	312.7	313.7	313.8	318.5	318.6	318.8
	4	279.1	280.3	380.2	284.6	284.4	284.7
	6	243.3	244.5	244.5	248.0	248.2	248.2
	8	206.5	207.5	207.4	210.4	210.7	210.5
	10	211.4	212.6	212.6	215.4	216.1	215.3

TABLE II.

	Section.	Rib Shortening Considered.			Rib Shortening Neglected.		
		$I=I_c$ sec. i. (2)	Ave'ge I (4)	True I (6)	$I=I_c$ sec. i. (1)	Ave'ge I (3)	True I (5)
Bending moment due to temperature. (Kip-feet.)	2	382.8	373.4	377.6	346.3	362.6	363.1
	4	680.6	663.8	671.3	615.5	644.6	645.5
	6	893.3	871.3	881.1	808.1	846.1	847.2
	8	1020.9	995.7	1006.9	923.5	967.0	968.3
	10	1063.4	1037.2	1048.9	962.0	1007.2	1008.6
Axial thrust due to temperature. (Kips.)	0	37.9	36.9	37.4	34.3	35.9	35.9
	2	38.9	37.9	38.5	35.2	36.9	36.9
	4	39.8	38.8	39.2	36.0	37.7	37.7
	6	40.4	39.4	39.8	36.5	38.2	38.2
	8	40.8	39.7	40.2	36.9	38.6	38.6
	10	40.9	39.9	40.3	37.0	38.7	38.8
Flange area required. (Sq. ins.)	0	68.38	68.59	68.71	69.40	69.59	69.65
	1	72.83	73.37	73.45	75.63	76.11	76.11
	2	78.09	78.47	78.58	81.97	82.79	82.80
	3	85.01	84.13	84.19	87.20	88.23	88.28
	4	90.74	89.41	89.59	88.93	89.83	90.29
	5	93.14	91.58	91.80	90.15	91.49	91.54
	6	94.85	92.99	93.23	88.01	89.61	89.64
	7	94.79	92.71	92.95	85.80	87.52	87.59
	8	92.25	90.13	90.39	82.12	83.10	83.11
	9	89.33	87.15	87.36	80.01	81.99	81.79
	10	88.97	86.81	87.05	78.24	79.94	79.63
Flange area used in design. (Sq. ins.)	0–2	80.22	80.22	80.22	82.50	83.61	83.61
	2–4	91.50	90.36	90.36	89.22	90.36	90.36
	4–6	94.89	93.72	93.72	90.36	91.50	91.50
	6–8	94.89	93.72	93.72	89.22	90.36	90.36
	8–10	92.58	90.36	90.36	82.50	83.61	83.61
Moments of inertia. (1000 in.^{4})	0–2	147.2	147.2	147.2	151.5	153.6	153.6
	2–4	168.3	166.2	166.2	164.1	166.2	166.2
	4–6	174.7	172.4	172.4	166.2	168.3	168.3
	6–8	174.7	172.4	172.4	164.1	166.2	166.2
	8–10	170.35	166.2	166.2	151.5	153.6	153.6
Maximum positive shear. (Kips.)	1	100.4	98.0	98.2	89.3	88.8	88.9
	2	89.8	87.6	87.8	79.6	79.2	----
	3	82.0	80.0	80.2	72.9	72.5	----
	4	76.7	75.1	75.2	68.8	68.4	----
	5	74.5	73.1	73.3	67.6	67.3	67.2
	6	76.1	74.9	75.0	70.3	70.0	----
	7	79.2	78.2	78.3	74.9	74.6	----
	8	81.1	80.6	80.6	78.2	78.1	----
	9	81.2	81.0	81.0	79.8	79.8	79.8
	10	79.6	79.6	79.6	79.6	79.6	----
Thickness of web. (Inches.)	0–2	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{7}{16}$	$\frac{7}{16}$	$\frac{7}{16}$
	2–4	$\frac{7}{16}$	$\frac{7}{16}$	$\frac{7}{16}$	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{3}{8}$
	4–7	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{3}{8}$
	7–10	$\frac{7}{16}$	$\frac{7}{16}$	$\frac{7}{16}$	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{3}{8}$

Second, that the flange areas when I is constant are practically the same as when the true values of I are employed. When the effect of rib shortening is considered the greatest difference for any section is less than 0.3 per cent., all the areas being slightly less when I is constant and equal to the average value of the true moments of inertia. This difference is so small that it did not affect the composition of the flanges in the two designs.

Third, that when $I = I_c \sec i$, and the rib shortening is considered, the required flange areas range from 99.1 to 102.2 per cent. while the required web sections vary from 100.0 to 102.2 per cent. of the true values. Excluding the stiffeners and splices which may be assumed as equal in the two cases, the amount of material used in the entire arch rib is 1.13 per cent. greater in the former than in the latter case. This includes the difference between the required sectional areas and those adopted in the designs.

It is seen therefore that the error introduced by the use of the formula $I = I_c \sec i$ has a relatively slight influence upon the design. Since the computations by this method are considerably simpler than those of the method of summation either in connection with a constant moment of inertia or with its true values, it appears that the former method is preferable.

SIMPLIFYING THE COMPUTATIONS.

The next step is to inquire whether it is not possible to still further simplify the computations by determining the horizontal thrust due to rib shortening separately rather than in combination with the thrust due either to the dead or the live loads by the complex formula referred to in connection with the second design. Since it is generally necessary, or at least preferable, to prepare a table from which the axial compression may be found for any given loading it requires but little additional labor to re-arrange the material in the table so as to show what the average axial compression is for any given loading. In the example used, the difference between the average of the compression in the twenty divisions and the true average compression throughout the rib must be very small. This method is used in Art. 80 of the text-book referred to in connection with the first design. The areas obtained by the two methods are given in Table III for comparison.

TABLE III.

Section	0	1	2	3	4	5	6	7	8	9	10
Design No. 2	68.38	72.83	78.09	85.01	90.74	93.14	94.85	94.79	92.25	89.33	88.97
Simplified method.	68.44	73.15	78.24	85.17	90.94	93.41	95.10	93.83	92.56	88.60	89.25

In the simplified method the thrust H due to rib shortening was computed from the unit stress derived from the average axial compression for the dead, live and temperature stresses combined. The rib shortening, however, modifies in turn this average axial compression. For instance, at the crown (section *10*) the thrust due to this first determination of the rib shortening is $H = -30.2$ kips. This causes an average axial tension of 29.4 kips, changing the average axial compression from 1705.0 to 1675.6 kips. The revised value of H is then found to be -29.7 kips, and the revised flange area to be 89.08 sq. in.

Attention is also called to the fact that in the formula used to find H in the second design, it appears that the square of the radius of gyration is assumed to be constant in those terms where it enters directly as a separate factor and not indirectly. It was previously stated that it varies from 6.371 to 6.393 with an average value of 6.388 while 6.38 was used. Therefore the flange area of 88.97 sq. in. at the crown is not quite correct. The correct value doubtless lies somewhere between 88.97 and 89.08 sq. in., and accordingly it may be concluded that practically the same results may be obtained by either of these methods. When the difference in labor is regarded there is no doubt as to which method of computation should be selected.

The flange areas given in Table III for the simplified method vary from 99.5 to 102.5 per cent. of the areas required in the sixth design, the former percentage being for section *0* and the latter for section 10. In most cases, therefore, it will not be worth while even to make the revision referred to in the preceding paragraph but one. In no case will it be necessary to revise the loading since the effect of such a revision is to give areas that differ more from the true values given in the sixth design, although they agree more closely with the areas of the second design. For example, in section *7* the revised area is 95.00 sq. in. or 102.2 per cent., and in section *9* the area is 89.50 sq. in. or 102.8 per cent. of the corresponding areas in the sixth design. While the term "true areas" has been applied to those of the sixth design it must not be forgotten that in the formulas used even in that case there is also a slight element of approximation on account of using the method of summation for a limited number of divisions instead of that of integration. The influence of this element is doubtless very small.

THE INFLUENCE OF THE RISE OF THE ARCH.

In order to show the influence of a change in the rise of the arch rib upon the relations discussed above for cases in which the ratio of rise to span was about one-tenth, three additional designs, numbered 7, 8, and 9, were made by Mr. Gibbs for an arch rib of the same span and loading but with double the rise, or about one-fifth of the span. The effective depth of the rib was reduced from five to four feet since the maximum moments are considerably less in the cases where the effect of rib shortening is considered. The conditions in these three designs are the same as those in the first, second, and sixth designs respectively.

In the seventh design where the rib shortening is neglected and $I = I_c \sec i$, all the moments are the same as in the first design except those due to temperature which are less than one-fifth as large. Compared with the same design, the axial thrusts due to the dead and live loads are somewhat more than half, while those due to temperature are less than one-tenth, as large. The relations of the eighth and ninth designs to the second and sixth designs respectively are about the same as those just given except those of the dead load moments. In the former the dead load moments are between one-fourth and one-fifth as great as in the second design, and in the latter between one-seventh and one-eighth as large as in the sixth design.

The flange areas required in the different sections, expressed in square inches, are as follows:

TABLE IV.

Section	0	1	2	3	4	5
Seventh design	40.26	47.83	53.98	58.26	59.23	58.89
Eighth design	40.23	47.41	53.32	57.85	60.57	60.39
Ninth design	40.25	47.86	54.10	58.39	59.52	60.00

Section	6	7	8	9	10
Seventh design	57.22	53.43	49.15	44.72	44.03
Eighth design	58.96	55.77	51.19	47.18	46.03
Ninth design	57.57	54.17	49.55	45.46	44.32

The thickness of the web in the eighth and ninth designs is $\frac{9}{16}$ inch in the first two divisions from the end, $\frac{1}{2}$ inch in the next division, $\frac{7}{16}$ inch in the next four divisions, and $\frac{1}{2}$ inch in the following three divisions to the crown of the arch. In the seventh design the required flange areas vary from 98.2 to 100 per cent. of the corresponding areas in the ninth design, while in the eighth design they range from 98.5 to 103.9 per cent. The flange areas finally adopted were the following:

Sections	0–2	2–4	4–6	6–8	8–10
Seventh design	54.00	59.25	59.25	57.75	49.50
Eighth design	53.25	60.75	60.75	59.25	51.75
Ninth design	54.00	60.00	60.00	57.75	49.50

Excluding the stiffeners and splices which may be assumed as equal in the two cases the amount of material used in the eighth design is 1.28 per cent. greater than in the ninth design.

It is seen, therefore, that this great increase in rise has served to reduce the effect of the rib shortening very materially and that for a rise exceeding one-fifth of the span it might be omitted in the computation. In view of the slight excess of material in the eighth over that in the ninth design, all the statements made in comparing the designs for the arch rib with a rise of one-tenth of the span may be applied to all intermediate values of the rise up to one-fifth of the span.

On comparing the material in the arch ribs exclusive of the stiffeners and splices, the ninth design requires only about 71 per cent. of the material needed for the sixth design.

The thesis of Mr. Gibbs from which the data in Tables I, II and IV are abstracted represents a very great amount of painstaking labor as well as good judgment in making the computations for the designs. The discussion in this article is extended somewhat beyond the limits which Mr. Gibbs set for himself, and the results of some additional computations are included.

STAND-PIPE AND PUMP-HOUSE FOUNDATIONS.

A. E. DUCKHAM, C.E., PITTSBURG, PA.

In the summer of 1898 it was decided to build a pumping plant to supply the engines and blast furnaces of one of the largest iron and steel firms in the United States with water from the Monongahela River. At first the capacity was decided upon as 24,000,000 gallons (in 24 hours), but later it was increased to 36,000,000 gallons—or two 6,000,000 gallon and two 12,000,000 pumps.

Originally the stand-pipe and pump-house were planned to be on one foundation; but, the contractor being afraid to take the risk of cracking from uneven settlement, it was subsequently decided to place them on separate foundations.

As shown on the plans, we figured on 225 piles under the stand-pipe and 688 under the pump-house. This was on the basis of 30,000 pounds, or 15 tons, on each pile. The calculated weights of the two foundations were as follows:

STAND-PIPE.

Water and iron in stand-pipe	1,405,850 lbs.
Stone and concrete	3,490,250 "
Wood in grillage	580,800 "
Total	5,476,900 lbs.

Dividing this by 30,000 (for each pile), we get 182 piles; but we made it 15 rows each way, or 225 piles for the additional pressure brought by the leverage of wind.

PUMP-HOUSE.

Oak in grillage	2,130,000 lbs.
Stone and concrete	18,240,000 "
Total	20,370,000 lbs.

This divided by 30,000 gives 679 piles, but to make the rectangle of the rows even, we made it 688. In both cases we neglected the buoyancy of the water, as being on the side of safety.

Figuring for the stability of the stand-pipe, we assumed the wind as concentrated at a point half way up and exerting a pressure of 30 pounds per square feet on the cylindrical surface. The pipe being 135 feet high and the foundations 43 feet deep, we have a lever-arm about the foot of the stone-work of 110 feet. The lever-arm of the masonry is 19 feet. Then $\frac{135 \times 14 \times 30}{2} = 26,300$ lbs. The weight of masonry

= 5,477,000 — 1,299,000 (weight of water displaced by foundation—allowing for buoyancy) = 4,178,000 lbs. Taking moments about the heel of masonry (or point of overturning), 26,300 × 110 × S — 4,178,000 × 19 = 0, or S (the factor of safety) = about 20.

The walls are of ashlar sandstone backed with concrete, the wells and canals being faced with vitrified brick laid in cement mortar; though, in the author's opinion, a Portland cement facing would at least have answered as well as the brick. On top of the piles is a grillage composed of 6 layers of 12 x 12 in. hemlock timbers and a layer of 3-inch oak planking on top of all to help render the caisson water tight while sinking, this latter being calked.

The piles were driven and cut off below water by means of a circular saw driven by an engine. Around the heads of these was deposited cement concrete, which was leveled by dragging a steel rail across the tops of the piles. Afterwards, before the caisson was completely sunk, a diver went down and examined the foundation to see that all was level and that no drift had gotten in. The caisson was built by fastening vertical 6 × 8 in. timbers to the grillage, letting them into the sides of the top timbers and spiking them fast. To these, two inch planking was fastened horizontally and calked. The sides were then braced across. The caisson was then floated and secured approximately in position by cables. Upon this the masonry was built and the caisson sunk by the weight of concrete and stone. When the caisson was within a few inches of the bottom, it was accurately lined into position and guyed securely until it touched the top of the piles. No difficulty was experienced while doing this part of the work, though the caisson was unusually large, being 50 × 150 feet, and very hard to brace inside, owing to the many openings—wells and canals. The braces inside, to prevent the pressure of the water from pushing in the sides of the caisson, were removed as the masonry was put in, and their places supplied by short struts between the sides of the caisson and stonework.

The "intakes," which supply the canals and wells with water for the pumps, are, as shown on the plan, 30 inches in diameter. Only half of those shown are to be used at present, the remainder being intended for a case of emergency; for example, if it is found that too much drift and rubbish from the river should get into the pump-house canals, a filter (made of wooden cribbing, stone and gravel) will be built in the bed of the river. It will be noticed that these last intakes are lower than the others for that purpose. All are closed by a half-inch cast-iron cap on the river side. The intakes now to be used will have the caps removed (they are only calked with oakum and clay, being held in place by the water pressure) ; the others will remain. These intake pipes of cast-iron have gates or valves (similar to those used on a water main) bolted to the inside flange ; as also have the pipes connecting the different sets of canals.

A "syphon-pipe" also runs the entire length of the plant, to be used with a jet of steam to pump out the water from the canals and wells, if it should became necessary to make repairs. In this case, the gates would first be closed.

All the water after entering the canals is screened or filtered by a screen of wooden (yellow pine) frame work made in four parts for each canal. These can be pulled out for repairs, sliding up like a window. The frame is covered with No. 11 galvanized iron wire with one-quarter inch mesh. The three feet next to bottom of canals are stationary and made of two inch plank spiked to the stationary frame or casing, and at the bottom to the grillage. This makes the screen come above the "mud line." As will be seen on plan, the bottom of wells and canals is the top of grillage. Arched openings admit the water to adjoining canals and wells where there are no gates.

Gratings, made of 2½″ × ⅜″ steel flat bars, with washer separators between and connected by ⅝″ steel rods supported on I-beams cover the canals and wells.

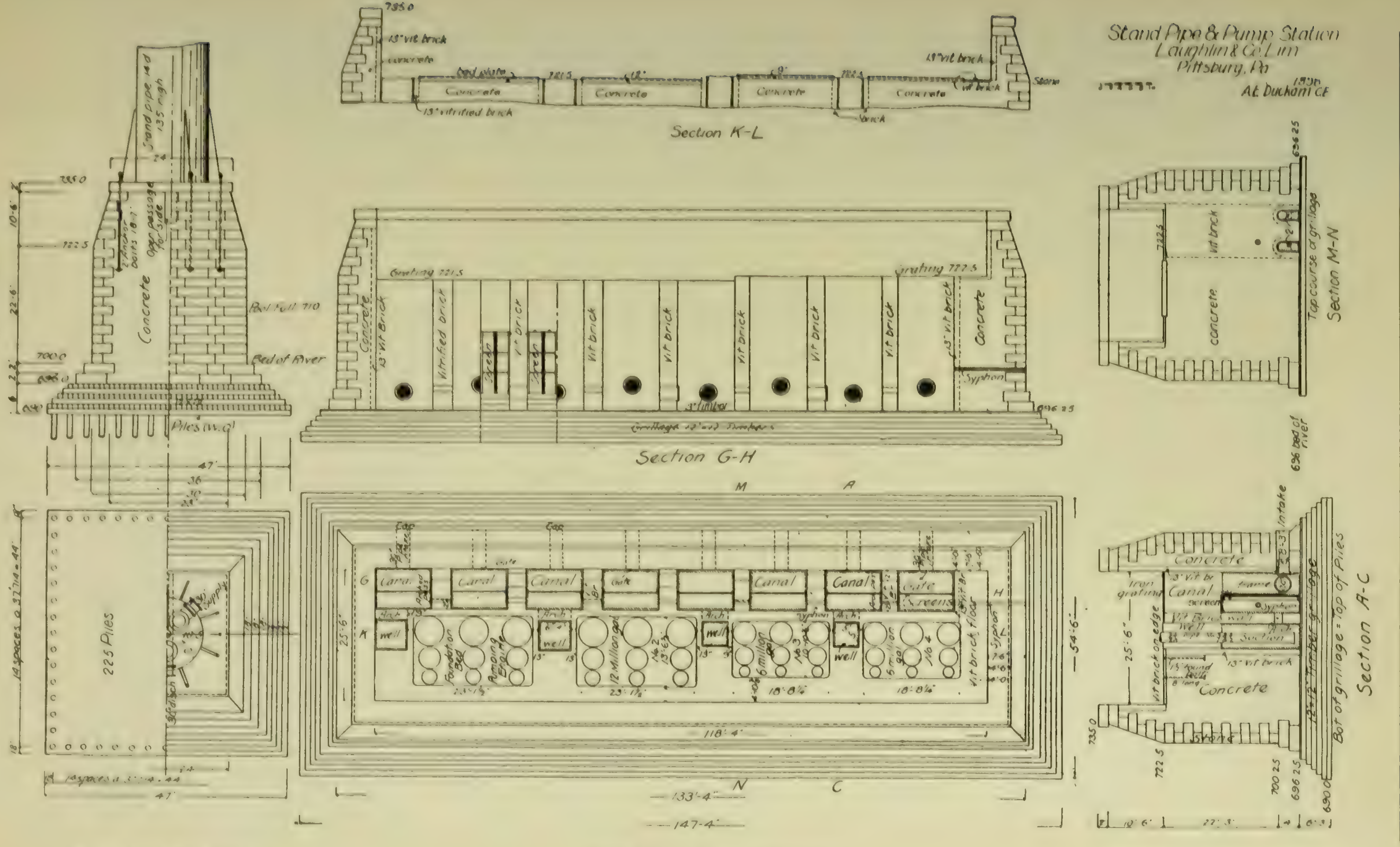
A "suction pipe," as shown, runs down into each well to supply the pumps with water.

It might be well to state that the high-water mark is at elevation 732 (above mean-tide at Sandy Hook), and that the normal height of water is 710 ("pool-full"); the Monongahela river being a navagable stream, kept so by dams. The site of the water works is above dam No. 1. The elevation of pool-full at Pittsburg, two miles below, is 704, or six feet lower.

The stone used was Beaver sandstone. The concrete was composed of American Portland cement, river sand and gravel, in the following proportions: 1, 3 and 9. The cement mortar for stone and brick work was mixed with sand 1 to 2. These gave excellent results, though a large part of the work was put in during the cold weather of this last winter. The work was under the eye of an inspector at all times.

The pump house is, of course, under roof. It is covered by a steel roof on steel columns, with brick "curtain walls" between. But, as our topic was *Foundations*, we will not concern ourselves with it.

It is to be hoped that this article may interest and be of some value to those engaged in similar work. If it does, the author will feel amply repaid for his time and trouble.



Plan of Stand Pipe Foundation

Plan of Pumping Station Foundation

THE PRESENT STATUS OF ENGINEERING KNOWLEDGE RESPECTING MASONRY CONSTRUCTION.

DAVID MOLITOR, M.AM.SOC.C.E.

I. INTRODUCTORY.

Masonry construction, in its relation to engineering, may be regarded as the origin from which the present science was evolved by a gradual progress of human research and experience. In the history of the world's advancement, it constitutes a measure of the degree of civilization. It is older than the written history of any people ; it ranks as the foremost instance in the adaptation of Nature's forces and products to the uses and convenience of man : and, as an engineering subject, it deserves the highest considerations, and gives birth to monumental works which, by their natural beauty and lasting qualities, stand as wonders of the world.

The volumes treating of the mechanics of engineering construction, or more particularly the subjects of earth pressure, retaining walls, and masonry arches, constitute a very voluminous library. It would require many years of patient reading to review only the most important German, French, English and American productions. However, to infer from this that our knowledge of the theories of masonry structures has advanced to a state commensurate to modern engineering science is entirely erroneous. On the contrary, this vast mass of literature represents in reality an unceasing effort to determine a few complex natural laws by an endless series of mathematical speculations based on a meagre assortment of experimental data.

The truth of this assertion must be conceded when it is remembered that all of our theories on earth pressure are derived from miniature laboratory experiments, generally made with sand or grain, and under conditions vastly different from those found in nature. To within four years ago experiments relating to masonry arches were limited to a very few of almost insignificant character. As problems of this nature can be solved only by empirical methods, it is natural to conclude that the empirical laws found apply only within the scope of the experiments. Hence earth pressure and arch theories based on small laboratory tests are quite worthless when applied to full size structures. Modern writers frequently treat of these subjects, but they advance nothing essentially new. Starting from the same erroneous assumptions employed by the old pioneers, they arrive at the same old conclusions, though the intermediate steps may be new and original. Hence such efforts do not represent real progress.

Much credit is due to the old theorists, among whom may be mentioned Coulomb, Poncelet, Scheffler, Culmann, Redtenbacher, Mohr, Considère, Winkler, Levi, Rankine, Weyrauch, and many others. They have practically exhausted the subject as far as the meagre experimental data available would warrant, and since the time of their investigations little or no progress has been made toward a satisfactory solution of the perplexing problem of earth pressure upon which the analysis of a large class of masonry structures depends.

Unlike the chaotic condition existing in regard to earth pressure theories, our knowledge of constructive details has steadily increased and many modern masonry structures stand as monuments to engineering genius.

Despite the test of time which the numerous engineering stone structures of past ages have endured, it is nevertheless true that our modern high grade masonry generally excels in quality and appearance the famous specimens of mediæval and ancient production. However, this is not generally true of the esthetic feature of modern designs.

Our modern, practical, improved methods of construction have never been excelled, or even equalled, and this feature in particular is entitled to even more credit as pertaining to engineering progress than would be a purely theoretical advancement.

While the Pyramids of Egypt are still recognized as a "world wonder", yet the conditions under which they were produced, if known to us, would probably permit of their construction now as well as in ages past.

The wonderful improvements made in recent years in the manufacture of cements is undoubtedly responsible for much of the success attending modern achievements in masonry construction.

The elaborate arch tests made in 1890–95 by the Austrian Society of Engineers and Architects demonstrate that fixed arches may, within certain limits, be designed and constructed by the methods hitherto employed without incurring more error than is inherent in the nature of the problem, but the inference frequently drawn from these tests regarding the applicability and reliability of past theory and practice respecting long span fixed masonry arches is somewhat erroneous. In fact these experiments show conclusively that fixed masonry arches invariably fail by flexure and not necessarily by compression, as under heavy loading the masonry breaks its continuity at the haunches and later at other points, creating flexible or hinged joints, and failure finally occurs by a collapse of the masonry at the points so weakened.

In the light of these tests, therefore, it would seem proper to provide artificial hinges, thereby making the masonry arch susceptible of rigid analysis, permitting higher unit working stresses, and obviating the danger of objectionable and often serious cracks generally occurring in large arches. This idea, after passing through various primitive stages in its development, finally resulted in the production of a perfect three-hinged masonry arch of which type numerous masonry and concrete bridges have recently been erected in Germany. They have proven a success and virtually mark a new era in masonry arch construction, as the introduction of hinges has removed the innumerable difficulties attending the computation and construction of large masonry arches.

In the following, the subject is considered more in detail under separate headings.

2. EARTH PRESSUE THEORIES.

THEORIES IN COMMON USE.

Classification of theories. These may be divided into two principal classes.

First, those in which it is assumed that when the retaining wall of an earth bank fails, a prism of the supported earth severs its connection from the bank and slides on a plane surface called the surface of rupture. This is called the "theory of the prism of maximum pressure", originated by Coulomb, Poncelet and Scheffler, and was solved graphically by Professors Mohr and Karl von Ott.

The second class is founded on the theory of conjugate pressure. The differential equations, representing the equilibrium of a particle of earth in the interior of a mass, are applied to any point, in this mass, and by integration, the total resultant earth pressure is found. This theory, proposed by Rankine, Levi, Concidère, St. Venant, Winkler, Mohr and Weyrauch, is called "the theory of earth pressure in the indefinite mass." The graphical solutions of Professors Culmann, Scheffler, Karl von Ott, Winkler, and Chas. E. Greene are based on this theory.

Assumptions. These theories are based on certain assumptions, and when applied to retaining walls give rise to contradictions which cannot be harmonized with facts observed in practice. While this is not surprising, especially when all the phases of the problem are considered together with the very limited experiments upon which the theories are based, yet it is very perplexing and often discouraging to the practicing engineer, who considers himself a member of a learned profession, when he begins to realize his inability to solve what appears to be a most simple problem in engineering mechanics.

The assumptions are as follows: 1. That a certain kind of earth possesses a certain constant angle of repose. 2. That when an earth bank is artificially constructed with a slope exceeding the angle of repose for that material, a certain prism will sever its connection from the remaining earth and slide into a new position of equilibrium. This surface of separation, called "the plane of rupture", is assumed to be a plane surface. 3. That the resultant earth pressure exerted by such a sliding prism against the back of a retaining wall acts upon that wall in a direction which may vary between the limits of being parallel to the surface of the retained earth, to making the angle of repose of the earth with the normal to the back of the wall. 4. That the resultant earth pressure has its point of application at a height above the base of the wall equal to one-third the height of the wall.

Accepting these assumptions, the plane of rupture and the resultant earth pressure become definite and determinable, when the question of cohesion is neglected.

It will now be shown that these assumptions are all materially erroneous, approaching somewhat the truth for dry sand, but departing therefrom for other materials, such as common surface loam, clay, marl, and mixtures of these with each other or with sand.

Angle of repose and surface of rupture. To prove the fallacy of assumptions 1 and 2, it will be necessary to review the results of experiments on high embankments given in a paper on "Landslides" by the writer, see Journal Association of Engineering Societies, Vol. XIII, No. I, January, 1894. This subject will be taken up later, and only a passing reference here will suffice for the present purpose.

A sliding embankment representing the actual conditions which obtain when earth is deposited at a steeper slope than is permissible, is shown in fig. 1, where the material was Turneri clay. From this and a number of similar experiments for embankments of different heights it was found that the angle of repose for the same material was not by any means a constant, but an inverse function of the height of the fill. Also, that the surface of rupture had no resemblance to a plane, but was practically an hyperbola in section. This fairly represents the disparity existing between results obtained from small laboratory experiments and actual cases as they occur in construction.

Direction of Pressure. The third assumption regarding the direction of the resultant earth pressure is probably nearer to the facts than any of the others, though the following considerations are offered: According to Rankine and others, this direction is parallel to the surface slope, and according to the majority of writers the resultant makes the angle of repose of the material with the normal to the back of the wall. The former theory is not rational because the direction of motion of the sliding prism is not governed by the direction of the surface of the slope but is largely dependent on the surface of rupture. The sliding prism glides on the surface of rupture and the direction of motion will be parallel to that surface until the resistance of the wall changes this direction. When this occurs, then the direction of the resultant pressure must of course make the angle of repose with the normal to the back of the wall, but in the instant after rest is restored the direction of this pressure returns to its former position. It is readily understood that the pressure is a maximum when there is absolutely no motion and that in the instant when motion occurs by partial giving way of a wall, the pressure changes its direction and diminishes in intensity, thus relieving the wall of a portion of its burden and establishing a temporary state of rest.

Hence, in designing a wall the direction of pressure should be taken for the state of rest when the effect will be a maximum, as otherwise a motion is necessary to permit the earth to establish a frictional resistance with which to diminish its direct pressure against the back of the wall.

Point of application. The fourth assumption, that the point of application of the resultant earth pressure is above the base of the wall by a distance equal to one-third the height of the wall, could be true only for a perfect liquid, in which the pressure increases with the depth below the surface and the distribution of pressure is represented by a triangle. Were the earth an inelastic solid then the point of application of the resultant would be at half the height of the wall and the pressure would be uniform and be represented by a rectangle. However, neither assumption is correct but the two cases cited are the extreme limits, and since earth is neither liquid nor solid, the point of application must be located somewhere between ½ and ⅓ the height of the wall above its base. Hence the pressure would be distributed according to some parabolic curve of the second or higher degree, with its vertex at the top of the wall.

The general condition being more nearly solid than liquid, the point is probably about 0.45 of the height of the wall above its base, varying between small limits dependent on the amount of moisture contained in the earth. This is substantiated by a few experiments, made by a French engineer, Leygue, published in Annales des ponts et chaussées, 1885, II, p. 788.

VARIOUS RECENT THEORIES.

Foremost among these may be mentioned the theory proposed in 1885 by the French engineer, Leygue, previously referred to in this paper, and one by M. Chaudy, in Memoires et Compte Rendu des Travaux de la Société des Ingenieurs Civiles, Decembre, 1895, discussed by G. C. Maconchy in *London Engineering* of August 26, 1898.

Leygue bases his theory on experiments performed by himself with sand and grain from which he determines values for the empirical variable coefficient in the equation $E = c\frac{g\,h^2}{2}$, in which, E is the horizontal component of the resultant earth pressure, g, the weight of one cubic meter of material, and h, the height of the wall.

The theory of Chaudy results in the same equation in which the empirical variable c is theoretically deduced.

All the older theories are based on this same equation and hence the recent contributions just mentioned differ from them merely in the values assigned to this variable coefficient.

If earth, or earthy materials in general, were possessed with properties similar to water in which friction between the particles is constant and differing only in degree for the case of liquids of different density and viscosity, then the above form of equation might be accepted as rational. However, it is a well established fact that earthy materials retain their shape by virtue of frictional resistance combined with cohesion between the particles. Hence the law of equilibrium must, of necessity, differ widely from that observed for liquids and in all probability the above equation is incorrect in form, which would explain the reason why none of the methods hitherto proposed are general in their application to all retaining wall problems.

Leygue shows experimentally on a small scale that the point of application of the resultant thrust is at a point varying between 0.38 h and 0.5 h above the base of the wall. To this extent he has revealed a new fact which corresponds well with modern views. Chaudy still insists on having the point of application at $\frac{h}{3}$ above the base of the wall.

A rational theory of retaining walls (so-called) was published by R. Iszkowski, in Oesterreich. Monatschr. fuer den oeffentlichen Baudienst of June, 1898. This theory is based on substitution of a prism of masonry having a resisting moment to overturning equal to the moment of a triangular earth prism included between the slope of repose and a vertical plane through the crest of the slope of a given embankment. This slope of repose is supposed to be such a slope as would result by filling earth behind a vertical wall which latter is suddenly removed thus allowing the retained earth to establish a natural slope.

The fallacy of the argument is essentially in assuming that a prism of earth can exert an overturning moment about an edge, which is impossible. This can only be realized by a solid of sufficient strength to resist its weight on one edge, and even in the case of masonry such a point is found near ⅓ the base from the compressed edge. The data for determining the surface of repose is such as may be found in older text books and could not rationally be applied in the manner indicated by the author. The value of E is again a function of $\frac{gh^2}{2}$. The paper is interesting, but unfortunately does not increase our knowledge of the subject.

A NEW THEORY OF EARTH PRESSURE.

General remarks. The difficulty attending the evolution of a new theory of earth pressure may be appreciated by the fact that so little progress has been accomplished in the past 25 years, despite the many efforts made to advance our knowledge on this obscure branch of engineering science.

While the method proposed in the following may be open to criticism, yet it is thought to embody the most recent ideas and experimental data available. The method is not claimed to be perfect by any means, but, being based on facts which are not much in error though at variance with previous assumptions, the best results attainable with the present status of our knowledge are expected therefrom.

The subject of earth pressure will not be definitely understood until some government; or institution of learning shall spend a few thousand dollars on large experiments, as was done by the Austrian Society of Engineers and Architects in 1890–95, to solve the mystery of fixed arches. A satisfactory solution of this perplexing subject is considered attainable if the proper method is employed, and, viewed in this light, the neglect to undertake its solution must stand as a blemish on the profession. In the meantime the best available data must be utilized to the best advantage.

Before entering on this subject it will be necessary to abstract the results pertaining to the surface of rupture and angle of repose as found from experiments by the writer, published in the Journal of Engineering Societies, vol. XIII, January, 1894. These experiments were incidental to construction work on the German government railroad Weizen–Immendingen, a section of which was projected and built under the writer's charge in 1887–1890.

The material consisted mostly of Opalinus and Turneri clays, of which about a half million cubic meters were handled in the construction of railroad embankments, varying in height up to 18 meters. These banks were all constructed with 1½ to 1 slopes, and in the course of a few months the material assumed a natural slope which differed for embankments of different heights. In passing from the artificial to the natural slopes, the material usually separated along a distinct plane of rupture, which, together with the final slope of rest, or angle of repose, was carefully measured. These aggravating mishaps, however, make up an extensive series of elaborate experiments, on which the new theory is based.

To increase the value of these experiments a description of the material used is here given.

Opalinus and Turneri clays are almost identical in appearance and in composition, and it would be difficult to distinguish between them were it not for the strata by which they are separated. The Opalinus clay, varying in vertical depth from 45–75 meters, is found in the lower Oolitic and immediately above the upper Liassic epoch. The Turneri clay, with a vertical depth of from 15–21 meters, is located in the middle Liassic immediately above the strata of the lower Liassic. Both clays contain iron, and, when the atmosphere and water have not come in contact with them, they may be of a steel gray or blue color. The usual color, however, is brown or that of ferric oxide, and the presence of a slight percentage of finely divided silica causes them to glisten. In the natural state, whether blue or brown, these clays are very hard, almost slaty, but, on being exposed to the atmosphere and water, they disintegrate into a fine powder which, when wet, presents all possible properties objectionable to the engineer. The weight of a cubic meter of this clay in a naturally damp condition is about 1900 kilograms.

The Angle of repose was found to be a variable depending on the height of the fill. The actual surface of repose is not a plane but a curved surface as shown in fig. 1. However, for practical purposes an average plane was taken. From the various embankments which were constructed in Opalinus and Turneri clays, and from the numerous slides which occurred in this material, the following data (which are probably near enough to the facts for all practical purposes) were collected.

Table showing relation between H and ρ, for Opalinus and Turneri Clays.

Height of embankment H	0—2*m*	2—5*m*	5—10*m*	10—13*m*	13—15*m*	15—20*m*
Angle of repose ρ	45°00′	33°45′	26°40′	24°00′	22°00′	18°10′
Corresponding slope	1 : 1	$1\frac{1}{2}$: 1	2 : 1	$2\frac{1}{4}$: 1	$2\frac{1}{2}$: 1	3 : 1
Tangent of ρ	1.00	0.66	0.50	0.44	0.40	0.33

Hence, for any given H the corresponding value of tang ρ may be taken from the above table.

As far as is known, this law applies to all materials varying only in degree, being less marked for sand or sandy loam than for clay. However, no experiments on large sand embankments are at hand to verify this supposition.

The surface of rupture is, not a plane, but, a curved surface whose normal cross-section is an hyperbola as is seen from the example shown in fig. 1.

Fig. 1 represents a cross-section of a slide showing the proposed slope or original condition of the embankment, the actual surface of repose after the slide had established permanent conditions of equilibrium, and the actual surface of rupture corresponding to this surface of repose.

Let A B C be a given embankment, which, by virtue of the steepness of the slope C B, slides into the position represented by the dotted curved line D E K. The sliding takes place on the *surface of rupture* D K. After complete equilibrium is established between the embankment and the moving mass, the surface line A B assumes the position D E, while the slope line B C takes the irregular curve E K, *the actual surface of repose.*

The portion A D M, left standing, will have no influence on the sliding mass, and the level of the embankment is practically changed to D E. The depression from A B to D E could of course be filled up and thus cause the masses to slide farther and to comply with the conditions imposed by the extra loading. The load A B D E would also drop to some new position intermediate between A B and D E, and, after a number of repetitions of this process, the conditions of the equilibrium for an embankment of height O A might be found, but this would lead to the same general law that obtained by assuming the portion A D M removed.

For practical and theoretical reasons, the actual curved surface of slope is assumed to be replaced by a plane N K. This will produce little or no difference in the results and will conform with the usual manner of grading. The angle N K O which the plane N K makes with the horizontal $K\,O$ is called ρ. This angle is found to vary with the height $O\,D = H$.

It was found, from numerous slides, that in this material (Opalinus and Turneri clays), the ratio $\frac{DN}{OD} = \frac{m}{H} = \frac{1}{3}$ nearly.

Hence the distance

$$OK = L = m + \frac{H}{\tan.\rho} = H\left(\tfrac{1}{3} + \frac{1}{\tan.\rho}\right) \qquad (1)$$

Taking the horizontal line $O\,K$ through the foot of the slope, and the vertical $O\,D$ through the point of rupture D at the crown of the fill, as axes of co-ordinates, an equation representing the surface of rupture may then be found.

Assuming the curve to be an hyperbola whose asymtotic axes are parallel to the axes $O\,K$ and $O\,D$, its equation must be of the form:

$$(x + a)\,(y + b) = c \qquad (2)$$

where a and b are the perpendicular, off-sets between the asymtotes and the axes $O\,K$ and $O\,D$, and c is a constant depending on the eccentricity of the curve.

Taking the co-ordinates of the curve $D\,K$ from the figure, substituting these in the above eq. (2), then solving, from each successive three equations, for a, b and c, and averaging these results, values are obtained which may be substituted in eq. (2).

It is found, from numerous curves of rupture and for various heights H, that these constants bear the following approximate relations to H and ρ; or, indirectly, to L, since $L = H\left(\tfrac{1}{3} + \frac{1}{\tan.\rho}\right)$.

$$a = \frac{L}{5},\quad b = \frac{H}{5},\quad \text{therefore } c = \frac{6\,H\,L}{25} = 6\,a\,b \qquad (3)$$

Solving the above eq. (2) for y, and substituting from eq. (3), for $c - a\,b$, its value $5\,a\,b$,

$$y = \frac{5\,a\,b - b\,x}{a + x} = \frac{H\,(L - x)}{L - 5\,x} \qquad (4)$$

Table of Comparison of Observed and Computed Values of y for Diagram in Fig. 1:

Assumed x	0	2	4	6	8	10	12	14	16	18	20	22	24	26	28	30	31.5
Observed y	12.2	9.0	6.7	5.3	4.2	3.2	2.7	2.2	1.8	1.4	1.1	0.9	0.7	0.5	0.3	0.2	0.0
Computed y	12.2	8.7	6.6	5.1	4.1	3.3	2.7	2.1	1.8	1.4	1.1	0.9	0.6	0.4	0.3	0.14	0.0

The curve of rupture may then be plotted for any case in question provided H and ρ are given.

The above conclusions seem to apply to most other materials except as to the degree of curvature of the surface of rupture which for sand would approach a plane surface, as is indicated by small experiments, although not much reliance can be placed on any such conclusions unless they are based on the actual behavior of large masses of sand, for which no experiments have as yet been made.

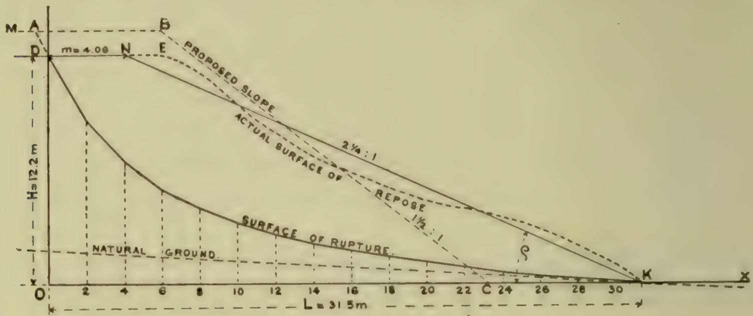


Fig. 1

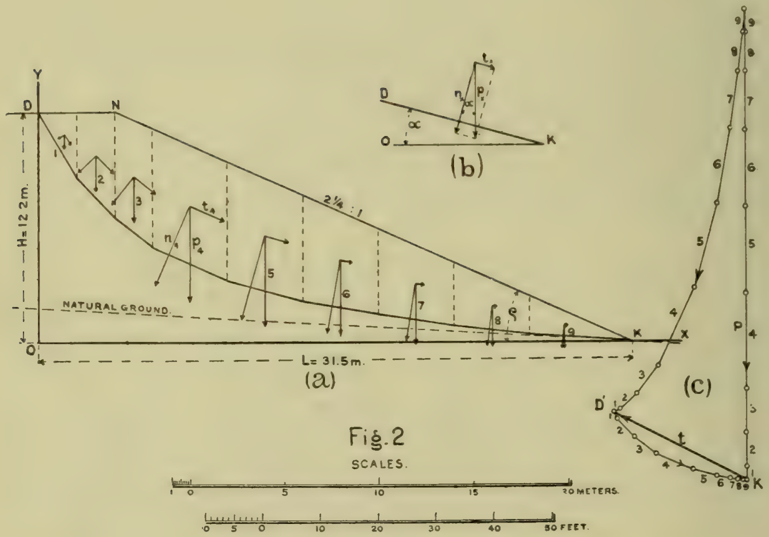


Fig. 2

Time is also an important factor in deciding on what actually takes place when a mass of earth is deposited for the purpose of obtaining its angle of repose. It may require six or eight months for a slope to change from its original to its stable condition, and if the difference between the two is but slight, it may require years. In the latter case it may take place by gradual settlement without showing sudden motion.

It will be observed that the term "angle of repose" is here used to signify the natural slope which a certain material acquires when exposed to the elements, and hence is dependent on the friction and cohesion existing between particles of earth. The term as commonly understood applies to friction between particles of loose, dry earth without cohesion.

The conditions of equilibrium for a certain mass of earth, as determined experimentally, are illustrated in fig. 1. Accordingly, any load imposed on the top of the embankment along D E, or on the slope near E, will disturb the equilibrium of the sliding prism and will cause the latter to move to a new position of rest. Also any additional mass placed on the slope near the point K will hinder sliding. Hence, in the nature of the case, the sliding prism, as determined in the preceding, represents the limiting case of equilibrium resulting from the balance between gravity forces and internal friction and cohesion of the material.

Therefore, the sliding prism D N K in its limiting condition of rest, exerts a tendency down the surface of rupture D K just equal to the maximum resistance t which the underlying earth is able to offer against sliding along this surface of rupture; therefore, any additional material added to the prism D N K will cause an increased tendency down the surface D K, which cannot be balanced by the resistance t, and motion will ensue.

This frictional and cohesive resistance t, when determined, will represent the maximum total resistance which can be exerted on any mass of earth between the foot of the slope K and the top of the bank at N, and any material in excess of the prism D N K will exert a downward tendency on the surface D K which must be resisted by a wall at K.

To determine the frictional resistance t, necessary to hold in equilibrium the prism D N K, it is necessary to find the resultant tendency of this prism down the surface D K for the limiting case of equilibrium (see fig. 2, in which a section of unit thickness is considered).

If the area of the sliding prism D N K be divided vertically into laminae or sections, then the weight p_x of each section will have a component t_x parallel to the surface of rupture immediately below the section and another component n_x, normal to this surface. In the limiting condition of equilibrium this parallel or tangential component t_x, representing the downward tendency on an inclined plane, fig. 2 *b*, must be exactly balanced by the frictional and cohesive resistance existing along the surface of rupture DK. This is only true in the limiting case. Hence the resultant of all the tangential forces along the surface of rupture will represent the downward tendency of the sliding prism. This resultant must be equal and opposite to the resistance t, which latter is found graphically in fig. 2 *c*.

To find the earth pressure on the back of a retaining wall, fig. 3, it is only necessary to determine the downward tendency T of the sliding prism $DSVK$, on the surface DK, in the manner indicated for finding the frictional resistance t, then knowing the direction and magnitude of t and T, they may be made to form a triangle in which the resultant earth pressure E is the unbalanced force, fig. 3 *b*.

Hence, the direction and magnitude of the resultant earth pressure are found directly and its point of application is taken at $0.45\,\overline{KV}$ above the point K in accordance with the previous discussion.

Another problem, without surcharge, is solved in fig. 4, where the earth pressure is also found graphically by the method given by Prof. Karl v. Ott, in which the area of the shaded triangle is sq. meters, multiplied by the weight of 1 cu. meter of the material, ($g = 1900$ klg.) gives the resultant earth pressure acting at $\frac{\overline{KV}}{3}$ above K. For this solution $E' = 21.7\ g$, agreeing closely with the result found by the new method except as to point of application.

In all of the above solutions it is advisable to use areas only instead of actual weights and in finding the opposing moment offered by the masonry wall, the area of the wall is multiplied by $\frac{g'}{g}$ to reduce to equivalent mass of earth. In the above this ratio was taken at 1.16 by assuming $g' = 2200$ klgr. for masonry.

The direction of the earth pressure as found, bears no relation to the direction of the back of the wall and this is as it should be, giving the most unfavorable condition. However, the resultant thrust in being resisted by the wall must necessarily be deflected in the instant when motion occurs either of the earth or the wall, but whenever a perfect state of rest obtains, then the thrust probably returns to its original direction of maximum effect.

A PROBABLE FUTURE SOLUTION.

From the above it is seen that the older theories are based on assumptions which are largely erroneous and while the truth may be more closely approximated in the light of recent experiments and ideas, yet no theory has as yet been formulated which may be regarded as perfect. Therefore, the time has come when a less speculative and more accurate method would be highly appreciated.

The simplest problem of a retaining wall, mathematically treated, leads to quite complicated equations and in general the graphical treatment is far more satisfactory, being more readily adapted to complex cases and affording a better check and insight into the otherwise obscure analytical methods. For this reason more success may in future be expected from a graphical treatment.

However, to reach a satisfactory solution, the nature of the materials involved, the many variable conditions affecting their behavior, and the prime importance of the subject, demand far more than mere mathematical investigation. The only promising solution will probably consist in an experimental or deductive determination of the actual pressure found on the back of a wall under various conditions.

It may seem vain to attempt the exact determination of a function depending on such variable material as earth, yet for a given kind of earth in a given state of moisture, a definite law certainly exists and this law once determined for different conditions of the same earth would enable the engineer to treat the problem intelligently and to estimate the probable error of his assumptions. This is by no means possible at the present status of our knowledge.

While the problem is not considered beyond the possibility of solution, the only reason for a failure to accomplish this end has been a lack or funds and a well developed plan of carrying on experiments. All engineering structures should be so designed as to comply with the most unfavorable conditions which may at any time occur consistent with economical requirements, and this in itself is sufficient reason why experiments and investigations should be made under corresponding unfavorable circumstances.

Of all the experiments made, or to be made, those only are valuable which are conducted under the actual conditions existing in nature and of these the extreme cases are most valuable in leading to proper conclusions.

Experiments performed on dry earth or sand are of little or no value since nature always supplies a certain amount of moisture which at times may increase to the limits of saturation, thus vastly altering the characteristics of the material.

The effect produced by moisture on earth, or sand, has been ascertained to be about as follows: The specific gravity increases with moisture ; the friction between the particles is a maximum for the dry state and diminishes with increased moisture, becoming almost nil for the state of saturation ; the cohesion is increased for a slight degree of moisture, but is diminished for a further increase and is nearly destroyed for the case of saturation. The earth pressure generally increases for increased moisture and becomes a maximum for the saturated condition, which latter may, however, be prevented by supplying proper drainage to the back filling.

It seems proper here to suggest a method for the successful solution of the problem of earth pressure which, at present, constitutes the most deficient branch of engineering science. As was said before, this may be expected only from an elaborate series of experiments to be conducted on a plan something as follows :

A very rigid wall should be constructed about 20 ft. high and perhaps 100 ft. long with good foundation on level ground. In a vertical section near the central portion of this wall a series of pressure gauges should be placed so as to communicate the earth pressure to indicators on the front of the wall. These pressure gauges would require special design in order that they may fulfill all the necessary requirements. The pressure surface might then be covered with a flexible oiled canvas to prevent the earth from squeezing in between the gauges and the masonry.

The pressure exerted by a fill, placed back of this wall, could now be observed for an extended period of time under various conditions of moisture, etc., and when all desirable measurements were made, the material could be removed and another kind of earth used to continue the experiments. This should be repeated until the variety of material used and the different heights employed would cover the subject in a general way.

The funds for such experiments could probably be obtained through an appropriation from the national or state goverment or from the National Society of Civil Engineers, and the work could be best performed under the direction of some large institution of learning. It is needless to mention the value which such an investigation would have for the profession at large, and the engineering students, participating in the work, would gain a very valuable experience.

3. DESIGN AND CONSTRUCTION OF RETAINING WALLS.

Without attempting to enter into the details of computation involved in ascertaining the stability of retaining walls and which may be found in books on masonry construction, so far as theory is applicable to the subject, it is intended here to consider more particularly the economic features of design in connection with practical considerations by which safety and durability in construction may be attained.

Economy in design. In considering a wall of given cross-section the stability is attained by a certain necessary mass of masonry which, by its resistance to overturning, must sustain the pressure of the retained earth. The quantity of masonry necessary to fulfill this requirement is, however, entirely dependent on the manner in which the mass of masonry is distributed over the section. That is, a certain earth pressure may be sustained, with equal safety, by walls in which the quantity of masonry employed may differ by 100 per cent., representing comparative economy in design.

The important effect of the shape of a wall on economy of material, is clearly illustrated by the following table in which certain relative dimensions have been worked out for different batters according to Rankine's theory by Prof. E. Haeseler. In each case the back filling is assumed level with the top of the wall and the angle of repose of the material is taken as 33°. The bottom width of the wall, b, as also the area of cross-section, A, are expressed as functions of the height of the wall, h.

[illegible] of earth

No. of Section.	Batter of		$g = g'$ and $\rho = 33°$.		$g' = 1.25g$ and $\rho = 33°$.	
	Front Face.	Rear Face.	$b = f(h)$.	$A = f(h^2)$.	$b = f(h)$.	$A = f(h^2)$.
1	Vert	Vert.	0.350	0.350	0.320	0.320
2	10:1	Vert.	0.327	0.277	0.300	0.250
3	5:1	Vert.	0.307	0.222	0.287	0.198
4	5:1	10:1	0.238	0.199	0.215	0.174
5	Vert.	5:1	0.472	0.372	0.456	0.356

This table, which is quite useful in taking off preliminary values, shows that section No. 4 is most economical while No. 5 is least so. The effect which the shape of the cross-section bears to the economy in amount of masonry required is also illustrated by the two solutions given in figs. 3 and 4.

From the above it is seen that the most economical form (local conditions permitting) would be one in which the wall is battered towards the retained earth with a batter just sufficient to retain the center of gravity of the section within the base of the wall, this limit being desirable for safety during construction, though, if care is exercised, the batter may be increased somewhat beyond this limit.

Provisions to insure safety and durability of retaining walls. These are applied in accordance with the causes of failure which will be briefly discussed before treating of necessary remedies for their prevention.

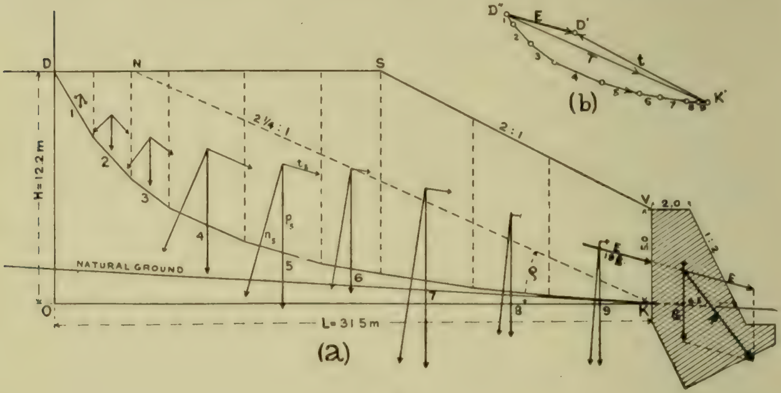


Fig. 3.

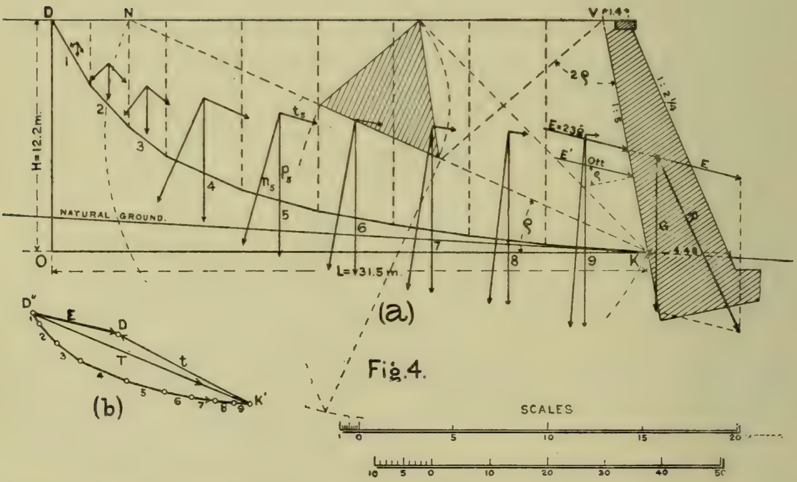


Fig. 4.

In computing the stability of a wall, it is customary to choose a section in which the resultant, of the earth pressure and the weight of the wall, shall remain within the middle third of the wall, thus striving to prevent any tensile stresses in the masonry. The entire question of stability is, however, based on the assumption that the foundation is perfectly rigid, which it never is except when bed rock is available. The yielding of the foundation is one of the most frequent causes of failure, hence the apparent necessity of the greatest precautions on these lines. Besides giving rise to local settlement in the alignment, a faulty foundation usually results in failure by partial or complete overturning of the wall.

Overturning may occur even on good foundations, and is then the result of poor drainage of the back filled material or scanty dimensions or poor masonry.

There are few, if any, long retaining walls which are free from vertical cracks, which latter may result from local settlement, excessive pressure or wide changes in temperature. The nature of the cracks will generally disclose their cause. A vertical crack of uniform width is usually the result of temperature stresses. A vertical crack, wide near the ground, gradually diminishing in width and disappearing at the top of the wall, is invariably the result of local settlement. Horizontal cracks are of rare occurrence, except in combination with vertical cracks, and generally have little significance. Changes in alignment always indicate overturning and usually accompany settlement. It is scarcely necessary to add that any or all of these causes may at times manifest themselves in a single effect, making a diagnosis more or less difficult.

When a retaining wall is founded on anything other than bed rock it is advisable to widen the base of the foundation masonry as indicated in figs. 3 and 4, so as to bring the resultant thrust *R* well within the central portion of the base. In doubtful cases the base may be widened sufficiently to allow the thrust to pass through its center.

Great care must be exercised in back filling a newly completed wall, since at this time the mortar is less capable of resisting stress than at any future time when the cement has become thoroughly set. Also, loosely deposited earth, possessing little cohesion in this state, exerts a greater pressure than when well compacted. Hence the back filling should not be applied until the mortar is well set and then the former should be deposited in successive layers, each of which should be well rammed.

As moisture always accumulates in the back filling, it is essential that proper drainage be provided adjacent to the wall, otherwise a condition of saturation may follow which would increase the earth pressure to its maximum value $\frac{g h^2}{2}$. This is about twice the pressure which any wall is usually designed to carry and will almost invariably prove disastrous.

The best and cheapest way to establish drainage is to provide a masonry gutter along the back of the wall at the lowest level suitable for draining either through the wall (by leaving openings at intervals), or at the ends of the wall ; preferably the former. From this longitudinal gutter, a dry rubble backing, of one to three feet in thickness, should be carried to near the top of the masonry, thus intercepting all seepage and completely draining the back filling. In severe cases, numerous rubble filled gutters may be extended into the back filling at right angles to the main wall. This is one of the most important precautions necessary in retaining wall construction.

To prevent vertical cracks from temperature effects in long walls, it is desirable to insert vertical expansion joints at intervals of 30 to 50 feet. These joints should be so constructed as not to weaken the resisting power of the wall, which is accomplished by making the joints dove-tailed in plan. No such provision is commonly made and the results are apparent when it is added that the linear coefficient of expansion of masonry ranges between one-fifth to one-third that of steel.

Cracks resulting from temperature effects should never be closed by filling in cement mortar unless it is found that they do not close in summer after long continued warm weather, in which case they were probably caused by shrinkage in joints and may then be filled.

A retaining wall which shows signs of overturning may generally be strengthened by the construction of buttresses placed at intervals commensurate with the case in hand, provided there is sufficient clearance in front of the wall to permit of this. A new wall may show indications of trouble and if supported by timber braces for a time sufficient to allow the back filling to settle and cohere in itself, the wall may become perfectly safe.

What has been said in the above regarding failures in walls applies generally to wing walls, masonry culverts, and abutments.

A new style of retaining wall might be employed with advantage, especially where a good foundation is not available.

The writer has frequently considered the advisability of constructing a wall of concrete and old railroad iron in such a manner as to make the back filling assist in producing the stability against overturning, see fig. 5.

The earth pressure is resisted by the wall, A B, acting as a beam in cross bending, being internally stiffened by old iron rails, or similar material, embedded cross-wise in the concrete near the outer or tension face of the wall. Stability is established by the rails G K acting as tie rods and held by the concrete base C D, which in turn is loaded with the retained earth. To take up the stress existing along B C D, the base C D is connected with the wall by occasional columns of concrete.

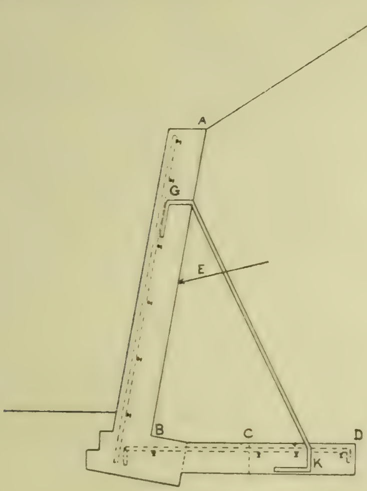


Fig. 5.

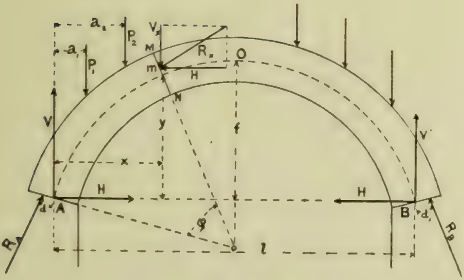


Fig. 6.

So far as known this idea has never been carried out, and it may deserve consideration in certain cases where local conditions are favorable to its application.

4. FIXED MASONRY ARCHES.

GENERAL CONSIDERATIONS.

The fixed arch, as distinguished from the hinged arch, belongs to the class of indeterminate structures in which the abutment reactions cannot be expressed in terms of the external forces without involving the elastic properties of the arch material.

In applying the theory of flexure to material like masonry, considerable doubt may justly be expressed as to the reliability of results obtained. However, the extensive arch tests conducted by the Austrian Society of Engineers and Architects, from 1890–95, have demonstrated very conclusively that stone, brick and concrete arches follow the laws of elasticity and the application of the theory of elasticity to such structures would seem perfectly justified in the light of these experiments.

Past experience has shown that only very low unit stresses are allowable in fixed masonry arches, a fact which proves beyond doubt that such arches are not adapted to very economical utilization of material. The reason for this is apparent. The initial stresses in portions of an arch, resulting from distortions and abutment displacements during and after construction, may often attain dangerous values, leaving only a small fraction of the breaking strength for a working stress. Fortunately, a partial failure of a portion of an arch, under excessive stress caused by distortions, has the tendency to readjust the distribution of stress by changing the shape to one better adapted to the strained condition. This is the reason why such distortions rarely become dangerous, but the fact illustrates the extent to which any theory is applicable to fixed masonry arches without using a large factor of safety varying from 10 to 20. Without this large uncertainty, resulting entirely from the rigid construction, high unit stresses of one-tenth to one-sixth the ultimate strength of the material might safely be employed.

Since masonry is not well adapted to withstand tension, no tensile stresses are allowed in arches. To accomplish this, the resultant thrust at any normal arch section must lie within the middle third of such section. This follows from Navier's theory for distribution of stress on a voussoir joint.

Hence, for a given span and load, the arch ring must possess a definite shape, requiring a very delicate adjustment of the line of thrust within the midde third of the arch ring.

Metal arches are independent of this requirement since the actual stresses arising from a certain loading and shape of arch can always be provided for when the dead load has been approximately assumed.

In designing a masonry arch, the shape, dimensions and weight must be known, or assumed, and if the resulting stresses exceed the allowable limits, then any or all of the assumed values must be altered until by a succession of approximations the proper shape and dimensions are found. This is a long and tedious process especially when the more accurate methods are employed.

According to the recommendations of the Austrian Society of Engineers and Architects, as a result of their elaborate tests, a fixed masonry arch should be constructed only when the following conditions can be realized :

1. The abutments must be perfectly rigid.
2. The falsework must retain its form during the period of construction of the arch ring.
3. The masonry must be of the best quality.
4. The construction of the arch ring must be most carefully conducted.
5. The falsework must not be released until the mortar has thoroughly set.
6. When the falsework is released, it must be done gradually and uniformly.

These conditions, except the two first named, can always be fulfilled, though the lack of rigidity of abutments and falsework are the two great obstacles in the way of long span, fixed, masonry arches.

ARCH THEORIES.

Historical. The earliest theoretical treatment of arches is ascribed to de la Hire who, in 1712, advanced the theory that an arch acted as a wedge in which the central or crown portion had the tendency to slide between the quarters adjacent to the abutments. Eytelwein, in 1808, applied this theory to individual arch joints and made allowance for friction.

The combined action of shear and bending was first recognized by Coulomb in 1773, and this theory assumes that an arch may fail either by one portion sliding on another, or by a joint opening as the result of bending. Experiments made by Boistard in 1808, proved that failures were to be expected only by cross-breaking and not as a result of shearing action.

Coulomb's theory was improved by Audoy (1820), Lame and Clapeyron (1823), Navier (1826), and Mery (1827), and was graphically treated by Poncelet in 1835.

The analogy between the arch and suspension bridge from which the theory of the line of thrust was evolved, was first established by Gerstner in 1831. This theory was improved and adapted to practice by Hagen (1844), Bauernfeind (1846), Schwedler (1859), Heinzerling (1869), von Ott (1870), Ritter (1876), and Wittmann (1878).

The theory of elastic deformation was introduced by Navier in 1826, by his analysis of the stresses on an arch section in which he assumes a combined thrust and bending moment distributed over the section. This theory was supported by experiments of Bauschinger and Koepke. Winkler (1867), and Belpaire (1877) apply this theory to the elastic arch, and the more recent modifications by Engesser, Ott, Mueller-Breslau, Melan, Weyrauch and others, are now generally accepted as the most reliable arch theories. They differ from each other only in methods of solution and degree of approximation. All are based on the same assumptions and the derivation of the formulae are essentially alike, but owing to the extremely complicated equations resulting from the theory, different authors have neglected certain terms supposed to be of insignificant value, while others, especially Melan and Weyrauch, attempt a rigid solution which is very complicated. The treatment of this subject by modern English and American writers is essentially according to Ott and Melan.

The graphical solution in general use is based on the most probable position of the line of thrust in a given arch ring for a given case of loading.

The theory of elasticity leading up to a purely analytical solution, is more reliable though far more complicated in its application.

The comparative accuracy and reliability of these two methods will be discussed in the following.

The theories in common use are two in number. First, *the line of thrust theory*, which is generally employed, and depends on an assumed, most probable position of the line of thrust within the arch ring; and second, *the theory of elastic deformation* by which the statically indeterminate abutment reactions are found by the application of the theory of flexure. The method of solution by the first theory is purely graphical, and by the second theory it is purely analytical and does not necessitate finding the line of thrust though this line is definite and determinable by the method. The second method may be made semi-graphical by the introduction of influence lines, which are very serviceable when dealing with concentrated moving loads.

To give a clearer idea of the distinctive features of these two theories it will be necessary to show the general relations existing between the external or applied forces and the internal stresses in a fixed arch. See Fig. 6.

The arch $A\,O\,B$ supports the loads P_1, P_2, etc., producing reactions R_a and R_b. The reaction R_a can be replaced by a horizontal component H and a vertical component V, acting at the point A, and a moment $R_a d = M$ about A. Also R_b can be replaced by H and V', acting at B, and a moment $R_b d_1 = M'$. Hence V and V' are the vertical abutment reactions, H is the constant horizontal thrust and M and M' are the abutment moments.

By taking center of moments about B and A, the respective values V and V' can be found thus:

$$V = \frac{1}{l}\left[M' - M + \Sigma^l_0 P(l - a)\right] \qquad (1)$$

and

$$V' = \frac{1}{l}\left[M - M' + \Sigma^l_0 Pa\right] \qquad (2)$$

For any section MN having coordinates x and y, the shear V_x bending moment M_x and resultant thrust R_x, are as follows:

$$V_x = V - \Sigma^x_0 P \qquad (3)$$

$$M_x = M + Vx - Hy - \Sigma^x_0 P(x - a) \qquad (4)$$

$$R_x = \sqrt{V_x^2 + H^2} \qquad (5)$$

$$N_x = V_x \sin\phi + H \cos\phi = \text{thrust normal to the section.} \qquad (6)$$

From N_x and M_x the stresses on the extreme fibres of the section MN may be found from

$$f = \frac{N_x}{F} \pm \frac{M_x}{W} \qquad (7)$$

in which F is the area of the arch section MN and W is the moment of resistance of this section.

It is readily seen that the determination of unit stresses resulting from any system of loading involves the terms M, M' and H which cannot be determined from purely statical considerations.

According to the theory of elastic deformation these unknown factors are determined from the elastic properties of the material, while in the older graphical solution they are merely approximated, by assigning certain assumed conditions to be fulfilled by the line of thrust.

Hence the theory of elastic deformation fixes the position of the line of thrust in terms of the elasticity of the material, while according to the graphical solution an infinite number of lines of thrust may be constructed for the same arch and same loading and it becomes a question to decide which of all the possible lines is the most probable.

Hagen (1862), according to his "theory of the most favorable distribution of stress", defines the most probable line of thrust as one for which the vertical projections of the mininum distances, between the line of thrust and the edges of the arch ring, become equal. Culmann (1866) designates this most probable line as the one which approaches most nearly to the arch center line in such a manner that the pressure at the most critical points becomes a minimum. Winkler defines the most probable line as the one for which the areas inclosed between the line of thrust and the arch center line are equally divided according to the method of least squares. Generally a problem is considered solved when, for a given span and load, a line of thrust is found which falls entirely within the middle third of the arch ring. This is, however, very far from constituting an acceptable solution as will presently be shown. Plausible proofs for all the above assumptions have long been sought but have never been found.

The graphical method is, at best, nothing more than the application of the principles of the three-hinged arch to fixed arches in which the location of the hinged points is arbitrarily assumed at the crown and springing. A line of thrust passed through these three points must remain within the middle third of the arch ring. If these points are assumed on the arch center line then M and M' become zero and H is determined as for a three-hinged arch. While all this is constantly done, there is nothing to justify the procedure.

The derivation of the formulae for M, M' and H is considered foreign to the purpose of this paper and is, therefore, omitted.

It is now proposed to show the comparative value of the two methods of solution of fixed arches and their reliability as found from the experiments of the Austrian Society of Engineers and Architects.

Reliability of the methods in common use. The graphical method, as was just stated, assumes that the real or most probable line of thrust is the one for which the highest pressures at the critical points becomes a minimum. Hence every line of thrust must either be the most probable line or else it is one which is less favorable than the most probable line. But, if only one line of thrust is possible for a given allowable stress on the extreme fiber at the critical points, then, if the structure is to be regarded safe for all time, this line must continue to exist when the material increases in strength (as by setting of the cement), or when the arch undergoes slight elastic or permanent deformations. For this there is absolutely no assurance, though it is an essential necessity in the general assumption.

It is also a matter of great uncertainty to know when any line of thrust really is the most probable line, since this depends on a more or less arbitrary definition, the requirements of which are fulfilled by a series of approximations.

From these and other considerations it follows that this method is neither reliable nor scientific. If arches, designed by this method, have stood well, it is most probably due to the extremely high factor of safety generally employed and not to the reliability of the method.

The analytical method, based on the theory of elasticity, is certainly scientific, though its value depends largely on experimental verification.

A sandstone masonry arch, of 23 m. span and 4.6 m. rise, was tested to destruction by the Austrian Society of Engineers and Architects. The breaking load was 3218 klg. per sq. m. on the half arch. For this load the formulae of Prof. Weyrauch give a maximum ultimate compressive resistance at failure of 33 atm. The sandstone had a compressive strength of 770 atm., and the cement mortar 311 atm. Samples of mortar, taken from the arch after destruction, broke under a pressure of 80 atm. The compressive strength of the masonry must, therefore, have been about 400 atm., or over twelve times the actual strength developed by the arch.

The results obtained from the tests on a brick arch and a Moniér arch, of practically the same dimensions as the sandstone arch, are similar to those just cited.

While it is not fair to expect very reliable results from the application of the theory of elasticity to material stressed to the point of rupture, yet the error incurred would probably not exceed 100% which would not begin to explain the still existing disparity in the above results.

It seems quite likely, however, that a very high initial stress existed in the arch as a consequence of the changes in shape of the arch ring resulting from the shrinkage of the mortar joints, by the setting process of the cement, after the mortar had attained sufficient strength to sustain the partial dead load imposed prior to removal of the centers.

If this supposition is justifiable, then it follows that though the theory may be correct and perfectly applicable within the limits of elastic deformation, yet the large factor of uncertainty resulting from permanent distortions, which must always be expected in fixed arches, necessitates a high factor of safety in addition to a very thorough mathematical investigation, if reliable results are expected.

While the theory of elasticity is undoubtedly the only trustworthy basis for the analysis of fixed arches, and should always be employed in designing structures of any importance, it does not follow that the older graphical method of investigation, and many of the empirical rules in common use, are all worthless. On the contrary, the preliminary design can be carried out most efficiently by the aid of the simpler methods, and the final design may then be tested by the application of the more accurate method.

It is thus seen that the theory of fixed arches has reached a status of perfection quite in keeping with the nature of the problem, the still existing uncertainty being a function of the material and other circumstances depending on the rigidity of the abutment foundations, conditions of erection, etc., all of which can never be entirely eliminated.

The above applies only to arches for which the loading is known with considerable accuracy. For arches sustaining high earth banks, the loading cannot be determined with any great degree of certainty. Hence the design of such structures is less definite, though the elastic property of the superimposed earth assists greatly in distributing pressure and rendering conditions more favorable to their safety.

The methods employed to obviate exeessive stresses in fixed masonry arches will now be discussed.

If it were possible to build an arch ring in such a manner that its line of thrust would pass through the center of the ring at the crown and haunches at the time of releasing the falsework, a large proportion of the indeterminate stress could be prevented. In other words, the bending moments at the critical points would then be almost zero for symmetrical loading, reserving the strength for the unsymmetrical live load and other contingencies affecting the shape of the arch ring.

In the ordinary process of construction, the arch ring is commenced at the haunches and the load gradually applied to the falsework, thus distorting the latter and causing bending stresses in the completed parts of the ring. The final settling of the ring produced by the contraction of the mortar joints as a result of compression and shrinkage during the setting process of the cement, even when the abutments remain perfectly rigid, are often sufficient to create serious initial stresses, thus destroying to a large extent the usefulness of the structure.

Several methods have been used, with more or less success, to relieve an arch of undue initial stress during construction.

One of the oldest of these is to set the arch stones on the entire falsework, spacing the joints by inserting small strips of wood and lastly filling all the joints simultaneously with mortar. This is still a very good program for erection, but is not very well adapted to large structures owing to the excessive stresses produced in the falsework and frequently causing considerable settlement in the entire arch ring.

This can, however, be prevented by wedging up the falsework just previous to filling the joints.

Another method frequently employed in the construction of brick and concrete arches is to close a complete ring adjacent to the falsework, and, after allowing the mortar or concrete to set, the remainder of the arch is constructed and the total load is carried by the first ring rather than by the falsework. By this means the settlement, during construction, is very slight, but, as is readily seen, the intrados is excessively stressed, a condition which cannot be averted and which is highly objectionable.

A program of construction frequently used on large arches, necessitates commencing work simultaneously at two, four or six points of the ring and closing at three, five or seven points, respectively. The longer the span the greater the number of points of commencement.

The most modern method consists in the introduction of temporary flexible or hinged joints at the crown and haunches and following the previous program of construction. These flexible joints are made of stone with curved, roller-like surfaces or iron blocks may be used, forcing the line of thrust through the center of the arch ring at the crown and haunches during the period of construction. These open joints are filled with cement mortar either before or after removing the falsework. Sheet lead has also been used for this purpose by inserting narrow strips into the crown and springing joints, allowing just sufficient surface of contact on the arch center line to carry the pressure without causing the lead to flow. The open joints are afterwards filled with good mortar.

This last mentioned method gives very good results in preventing cracks and excessive settlement during construction, but subsequent changes cannot be compensated for, which is the only obstacle still remaining in this class of structure. Eventual settlements in foundation masonry ; contraction of mortar by evaporation of water used in mixing the same ; compression of mortar joints from loading ; elastic deformations caused by temperature and load effects ; all these are ever present to create stresses which, in spite of all precautions during construction, may attain dangerous proportions and make it utterly impossible to estimate the actual ultimate strength of a fixed masonry arch.

It should be remembered that for masonry arches, the live load is generally only a small fraction of the dead load, and for this reason an arch which is sufficiently strong to sustain its own weight permanently will carry temporary live loads with perfect safety. When initial stresses attain breaking limits the masonry generally chips near the surface thus relieving the stress and allowing the line of thrust to return to a more favorable position. In this way an arch which has become distorted may readjust itself to a new condition of stress.

In conclusion it might be well to recall the recommendations proposed by the Austrian Society of Engineers and Architects, which, if strictly carried out, would limit the adaptation of fixed masonry arches to comparatively short spans and bed rock foundations, as these recommendations require the fulfillment of the following conditions :

1. The abutments must be perfectly rigid.
2. The falsework must retain its form during the period of construction of the arch ring.
3. The masonry must be of the best quality.
4. The construction of the arch ring must be most carefully conducted.
5. The falsework must not be released until the mortar has thoroughly set.
6. When the falsework is released, it must be done gradually and uniformly.

The use of sand jacks or sand pots, which was introduced in 1854 during the construction of the Austerlitz Bridge in Paris, offers a very novel and efficient means of releasing the falsework supports.

The necessity of these recommendations is clearly understood after what has been said regarding the theory of fixed masonry arches and its application to practice.

The first two conditions can only be realized when rock foundations are available and when the falsework is made rigid by very substantial foundations. The other requirements can generally be fulfilled by exercising proper care and by permitting only good material and workmanship.

5. THREE HINGED MASONRY ARCHES.

The extent to which theory is applicable to fixed masonry arches has been adequately demonstrated in the previous.

To obviate the difficulties and prevent the uncertainty shown to exist in fixed arches, the introduction of hinged joints at the crown and haunches may be regarded as a most welcome innovation, first applied by Koepke of Dresden, in 1880, by providing curved, open, voussoir joints in the arch ring at the crown and haunches. In 1885, Karl V. Laibbrand, of Stuttgart, substituted sheet lead for the open joints, and in 1893 applied cast iron, hinged bearings.

The three hinged arch is free from initial stresses otherwise resulting in fixed arches after releasing the falsework. Small abutment displacements, and changes in temperature, do not affect the magnitude or distribution of stress in the arch ring, and hence do not cause cracks.

This form of arch admits of a rigid analytical treatment. For this reason the allowable working stresses may be chosen much higher than for fixed arches ; they may range from $\frac{1}{10}$ to $\frac{1}{4}$ the ultimate strength of the masonry without sacrificing safety or durability. The strict requirement regarding the rigidity of abutments and falsework, so very essential to fixed arches, is a matter of comparative insignificance, provided the abutment foundations are safe.

Therefore, the three hinged arch is especially well adapted to the economic construction of long span bridges, and being free from the numerous objections inherent in the fixed arch, is well adapted to conditions where only moderately good foundations are available.

The introduction of hinges has placed the masonry arch on a high plane of engineering perfection, and has removed its restricted application, thus materially enlarging the field of usefulness so justly belonging to this most substantial, economic and aesthetic form of bridge construction.

No disadvantages have as yet developed from the use of hinges and none are likely to develop. Numerous bridges of this type, of spans ranging from 10 to 50 meters, have been constructed in Germany and Switzerland within the past 15 years. All are giving excellent satisfaction.

Therefore, owing to its economy, permanence, low cost of maintenance, and aesthetic features, the three hinged arch is destined to become a successful competitor of iron and steel bridges, in all cases where the natural conditions of foundations and length of span do not offer unsurmountable difficulties.

This subject has been exhaustively treated in a paper, read by the writer, before the American Society of Civil Engineers. (See Vol. XL, 1898, Trans. Am. Soc. C. E.)

6. CONCRETE AND IRON ARCHES.

This comparatively new system of construction, introduced about twelve years ago in France and Germany, is fast displacing the solid masonry structures of the past.

The desirability of combining concrete with steel or iron, evidently arises from the widely different properties of concrete under compressive and tensile stress. The compressive strength being about ten times the tensile strength (so far as our present knowledge of concrete would indicate) makes it impossible to use concrete with economy. In fact, common practice has always been to use concrete and masonry only under compressive stress. By supplying tensile strength to those parts of a structure where tensile stress occurs, by the insertion of steel or iron rods, the combined material becomes more widely applicable and is susceptible to a more economical utilization.

The various systems of concrete and iron arches (mostly patented), may be summarized as follows: The Monier, the Wuensch, the Moeller, and the Melan. There may be others, but they probably involve no new principles.

The Monier patent is so broad, embracing in a general way any iron parts enveloped by cement, that it is difficult to understand why not all other systems are infringements. However, this system as generally employed consists of a series of iron rods imbedded in the concrete near the surfaces of the intrados and extrados to supply the required tensile strength. The rods extend longitudinally from one abutment to the other and are tied together, at intervals, by wire. Expanded metal has been used in a similar manner.

The Wuensch system employs only longitudinal angle irons or tees, placed near the intrados and extrados of the arch, or beam, relying on the concrete to perform the functions of a web system.

System Moeller amounts to nothing more than a very flat concrete arch with a metal tie rod to take up the horizontal thrust. The tie rod is anchored directly in the concrete of the arch abutments.

The Melan system inserts concrete arches between iron *I* beams or latticed arches. It is, therefore, a succession of parallel metal and concrete arches in the same span.

The combinations of concrete with iron or steel, offer a great many advantages in practical and economic construction, though the application is perhaps too recent to warrant a definite conclusion regarding the lasting qualities of these composite structures. It must also be admitted that the theories of this class of structure are still very primitive.

Monier arches are generally designed for a constant modulus of elasticity

$$E = E_0 \left[1 + \frac{I_1 (E_1 - E_0)}{I E_0} \right]$$

in which E_0 = modulus of elasticity of concrete; E_1 = modulus of elasticity of iron; I = moment of inertia of the combined concrete and iron section; and I_1 = moment of inertia of the iron section alone.

This is based on the assumption that the two materials in acting together must undergo the same elastic distortions.

Arches built according to Wuensch, are designed to carry the total maximum load entirely by the concrete, and the unsymmetrical live load entirely by the metal parts. This is a somewhat arbitrary method.

The Moeller system probably admits of the most exact design and is especially well adapted to floor arches and light bridges. There is nothing about the design which would require special consideration.

The Melan arch being a combination of arches of different materials makes it difficult to determine the proportion of the total load carried by each material. However, since the elastic deformation of an arch is approximately proportional to the quantity $\frac{1}{EI}$, and two arches, if they would work together, must undergo the same deflection, it follows that the load must be divided in proportion to the products EI for each material, or that

$$\frac{\text{Load carried by the concrete arch}}{\text{Load carried by the iron arch}} = \frac{E_0 I_0}{E_1 I_1}$$

in which E_0 and I_0 apply to the concrete section and E_1 and I_1 to the iron section. This may be far from the truth, though it has generally given good results compared with tests.

Regarding the durability of concrete and iron structures we are not in a position to say definitely what the probabilities are. However, with the exception of the Melan system, the iron section is generally quite small and even a slight corrosion might prove disastrous because so much depends on the iron parts. It is quite likely that moisture might penetrate some of the hair like fissures generally found in masonry and concrete which would do mischief in the course of time. Roebling found such corrosion on some of the outer strands of the anchorage cables of the Niagara and Brooklyn bridges, where the cables had been embedded in concrete.

The widely different physical properties of concrete and iron do not seem to offer any serious obstacles as was at first predicted, at least none have developed during past experience. Concrete appears to possess an adhesive strength with iron which is equal to or even greater than the tensile strength of the former ; the coefficients of expansion are nearly enough equal in the two materials to offer no serious objections ; and by a proper distribution of loads their elastic properties may be so harmonized as to make them work fairly well together.

Meanwhile many new structures of this type are being constructed and time alone will suffice to clear up the still existing doubts. So much may, however, be predicted, that it is not unlikely that some of the very keen Monier arches, depending so largely on the slender iron network for their strength, are apt to prove unsatisfactory in point of durability. Melan and Moeller arches will probably be longer lived.

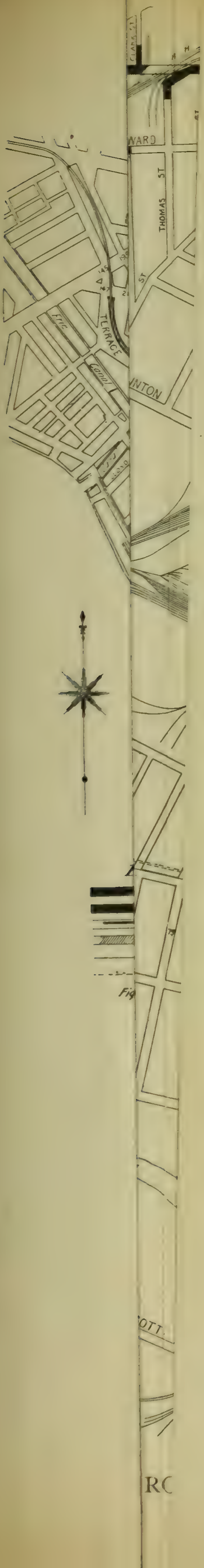
7. CONCRETE

Concrete has become one of our most important materials of construction and a few remarks may be appropriate in concluding this paper.

This material, universally used in compression, has been wonderfully improved in recent years by the general introduction of high grade cements, though our knowledge of the properties of concrete under compressive stress is rather meagre and much valuable work still remains to be done towards increasing our present supply of reliable experimental data.

Among the most desirable experiments to be made, the following might be mentioned: The determination of the shrinkage of concrete while setting from time of placing same till it has reached various ages, for different mixtures; the permanent set which concrete undergoes under various compressive stresses; the modulus of elasticity at various ages for different mixtures; and lastly the compressive strength of cubes corresponding to mixtures used for the above experiments.

Such work would be highly valuable and interesting as well. Those wishing to take up this subject may find some valuable suggestions in a paper by the writer on "The Properties of Concrete under Compressive Stress" published in the Journal of Engineering Societies, for May, 1898.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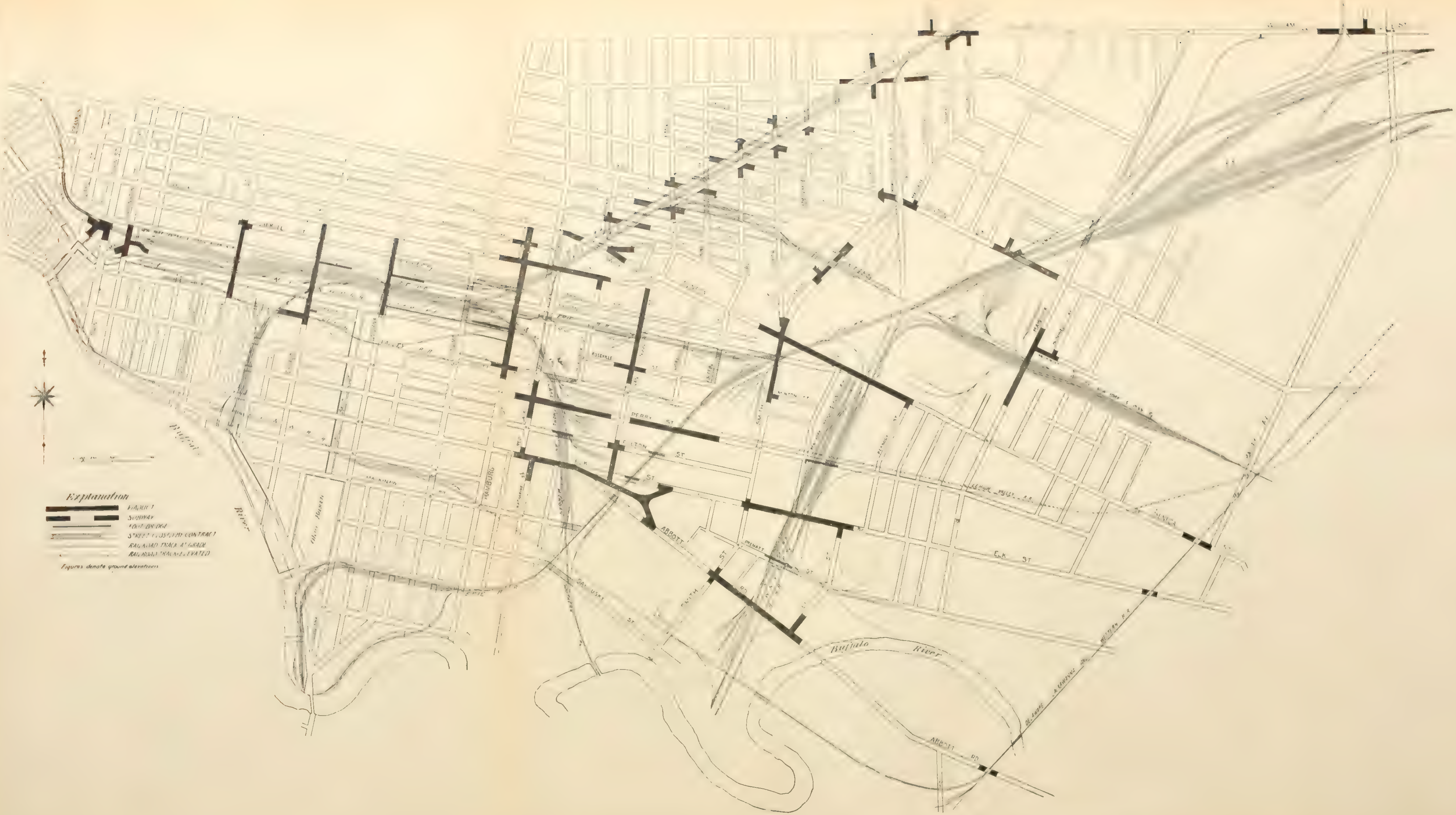

FIG. 1.—GENERAL CONTRACT PLAN—TRACKAGE IN 1892—BUFFALO GRADE CROSSING IMPROVEMENT.

THE ABOLITION OF GRADE CROSSINGS IN THE CITY OF BUFFALO, N. Y.

JOHN A. KNIGHTON, '91, ASST. ENGINEER.

The question of Grade Crossings and their abolition is one that doubtless is receiving more attention in this country today than it ever did before, and is destined to become even more prominent.

Three states, New York, Massachusetts and Connecticut, have passed laws placing in the hands of a Board of Railroad Commissioners power to institute proceedings for the change of any crossing at grade, either upon application from those interested or when, in their opinion, such change is necessary for the public safety.

Various cities have by mutual agreement with the railroads taken up the matter and accomplished much without resorting to legislation.

All that has been done in this country, however, is but little compared to what has been accomplished in Europe, where vast millions have been spent not in large cities alone, but in smaller towns as well.

The problem presented to the City of Buffalo was an exceptionally difficult one, on account of the various and varied interests involved, and again for the reason that it is impossible to lower either the streets or the railroads to any extent over a large area for want of drainage.

Buffalo is located at the foot of Lake Erie, and is the old transfer point between lake and canal. Later on railroads were built to share the carrying trade from lake to tidewater, and other lines were built to the west, with Buffalo still a transfer point.

The east and south sides of the City are comparatively level, and little above the elevation of Lake Erie, which is on an average about —3.0, and as the most direct line from either the east or the west to the business center of the City is through this section, it naturally became the railroad district, which in 1892 presented the appearance, so far as tracks are concerned, that is shown in Figure 1.

Buffalo has had a remarkably rapid growth, as shown by the following table:

Year	Population
1810	1,508
1820	2,095
1830	8,653
1840	18,213
1850	42,216
1860	81,129
1870	117,714
1880	155,137
1890	254,457
Present estimate	390,000

That the development of the railroad interests within the City has kept pace with this growth is shown by the fact that the proportion of City taxes paid by the railroads remains nearly constant or from $\frac{1}{10}$ to $\frac{1}{12}$ of the total. For the year 1898–9 the railroads within the City paid $458,670 out of a total of $4,601,021.

As early as 1856 a plan was prepared by the City Surveyor, Peter Emslie, for a bridge or viaduct over the tracks at Michigan street.

In 1874 the late George E. Mann, who was then City Engineer, designed a suspension bridge for William street over the N. Y. C. & H. R. R. R. tracks, and submitted it to Mr. Vanderbilt with a plea for its construction.

Neither of these structures were built, and Mr. Vanderbilt is said to have replied regarding William street "that until the City of Buffalo expressed a more emphatic desire for this improvement and they, the railroad company, were obliged to do the work, they would have to decline to consider the matter".

The first organized movement against grade crossings by the citizens of Buffalo was in 1887.

Delays and accidents at the many crossings were increasing and the various commercial interests of the City were suffering.

The Lumbermen's Exchange, the Merchant's Exchange, the Common Council and the press took up the matter in earnest, and on April 13, 1887, the Council passed the following resolution :

"That an invitation be extended to the said Railroad Commissioners to inspect the entire system of railroad approaches entering the City, with a view to securing their recommendation of a comprehensive plan for elevating the tracks over street crossings, or otherwise providing suitable remedies".

This resolution was forwarded to the State Board of Railroad Commissioners, which body was then composed of the following members, John D. Kernan, William E. Rogers, and Isaac V. Baker, Jr., as Commissioners.

A communication from the Buffalo Lumber Exchange, dated May 23, 1887, was also received by the Board, which called special attention to the crowded condition of the railroad crossings at Washington, Michigan, Chicago and Louisiana streets.

The Board held a public hearing in Buffalo on June 7, 1887, at which time the City in its corporate capacity, the Lumber Exchange and other interested parties were present, and before leaving the City the Board made a personal examination of the various crossings.

Almost immediately after, the Inspector of the Board, Mr. Thomas

) GRA

e street p

e street o

FIG. 1. GENERAL PLANS PROPOSED—BUFFALO GRADE CROSSING IMPROVEMENT

[illegible]	[illegible]										Grade										Under	L	R	Over	[illegible]
[illegible]	[illegible]										Grade										Under	L	R	Under	L [illegible]	R [illegible]
[illegible]	[illegible]													Under	L		[illegible]			[illegible]		St	[illegible]		
[illegible]	[illegible]	Under	[illegible]	R	[illegible]			[illegible]	Same as Spencer plan			[illegible]			[illegible]			Under		R	Over	L 8.9	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	L	[illegible]			Under			Grade							Under	L	R	Under	L [illegible]	R [illegible]
[illegible]		Grade									Grade							Under	L	R	Under	L 8.5	R [illegible]
[illegible]	[illegible]	[illegible]	L		[illegible]	L		Same as [illegible] plan			Grade			Under	L	R [illegible]	[illegible]			Over	R		Under	L [illegible]	R [illegible]
[illegible]	[illegible]																			Under	L	R	Under	L 7.87	R [illegible]
	[illegible]																[illegible] crossing at Elk street			Under	L	R	Over	R	
[illegible]	Erie	[illegible]									Grade						[illegible]			[illegible]			Foot bridge [illegible] street closed to teams at crossing at Elk street [illegible]		
[illegible]	[illegible]	Grade									Grade						[illegible]		L	Under	L	R	Over	R [illegible]	
[illegible]	[illegible]										Grade									Under	L	R	Under	L [illegible]	R [illegible]
[illegible]	[illegible]																			Under	L	R	Under	L [illegible]	R [illegible]
[illegible]	N. Y. C. & H. R.													Under	L	R 5.0	Under	L	R [illegible]	Under	L	R	Grade		
[illegible]	N. Y. C. & H. R.																			Under	L	R	Under	L [illegible]	R [illegible]
[illegible]	[illegible]																			Under	L	R	Under	L [illegible]	R [illegible]
[illegible]	N. Y. C. & H. R.																						Foot bridge		
Fulton	L. S. & M. S.																			Under	L	R	Foot bridge Street closed to teams		
Fulton	Erie																			Under	L	R	Foot bridge Street closed to teams		
Hamburg	N. Y. C. & H. R. W. S. Erie W. N. Y. & P.													Closed for under crossing			Under	L	R 8.0	Under	L	R	Over	R 16.80	
[illegible]	[illegible] W. S.																			Under	L	R	Under	L [illegible]	R [illegible]
[illegible]	N. Y. C. & H. R. W. S.																			Under	L	R	Under	L [illegible]	R [illegible]
Louisiana	N. Y. C. & H. R. W. S. Erie W. N. Y. & P.	Under	L 2.5	R	Over	R		Same as Spencer plan			Same as Spencer plan			Over	R	L 2.0	Over	R	L [illegible]	Under	L	R	Over	R 16.9	L 1.95
Louisiana	D. L. & W.	Grade															Under		R	Under	L	R	Grade		
Louisiana	Erie	Over	R		Same as Spencer plan			Same as Spencer plan			Same as Spencer plan						Over	R		Grade			Grade		
Main	N. Y. C. & H. R.	Grade crossing. Fence with gates either side of tracks						Same as Spencer plan			Same as Spencer plan			Grade	L 5.0	L 5.0	Over	R but little	L	Under	L	R	Over	R [illegible]	L 14.91
Michigan	N. Y. C. & H. R.	Under	L 2.5	R	Grade			Same as Spencer plan.			Same as Spencer plan			Over	R	L 1.5	Over	R	L 2.5	Under		R	Over	R 15.11	L 1.56
North Division	N. Y. C. & H. R. W. S. W. N. Y. & P.																			Under	L	R	Under	L 8.64	R 6.[illegible]
Oneida	N. Y. C. & H. R. W. S.																			Under	L	R	Under	L 7.69	R 7.68
Pearl	N. Y. C. & H. R.													Grade	L 1.5	L [illegible]				Under	L	R	Foot bridge		
Perry	L. S. & M. S.																			Under	L	R	Over	R 17.6[illegible]	
Perry	Erie																			Under	L	R	Over	R 15.1[illegible]	
Perry	B. C. N. Y. C. & St. L. W. N. Y. & P.																			Under	L	R	Foot bridge Street closed to teams		
Prenatt	B. C. N. Y. C. & St. L. W. N. Y. & P.																			Closed			Foot bridge Street closed to teams		
Seneca	N. Y. C. & H. R. W. S. W. N. Y. & P.	Over Under	R L 5.5	R 7.0	Over	R		Same as Spencer plan			Under	L 10.0	R 3.5							Under	L	R	Over	17.35	
Seneca	Erie	Grade									Under	L 8.75	R 4.75	Under	L	R 8.0	Same as Buchholz plan			Under	L	R	Over	R [illegible]	L 4.27
Seneca	B. C. N. Y. C. & St. L. W. N. Y. & P.	Over		L 2.5	Same as Spencer plan.			Under			Under	L 7.5	R 5.5							Under	L	R	Over	R 19.67	
Seneca	D. L. & W.	Grade																		Under	L	R	Under	L 7.2	R 8.17
Smith	N. Y. C. & H. R. W. S.																			Under	L	R	Under	L 7.65	R 7.72
Smith	Erie																			Under	L	R	Over	R 15.18	L 2.99
South Division	N. Y. C. & H. R. W. S. W. N. Y. & P.																			Under	L	R	Under	L 8.37	R 7.0
Swan, Seymour	N. Y. C. & H. R. W. S. W. N. Y. & P.																			Under In Seymour street Swan street closed	L	R	Under	L 11.42	R 3.95
Terrace	N. Y. C. & H. R.	Grade crossing. Fence with gates either side of tracks						Same as Spencer plan			Same as Spencer plan			Grade	L	L						R	Railroad cut and tunnel		
Van Rensselaer	Erie													Under		R	Same as Buchholz plan			Under	L	R	Over	R 18.17	
Washington	N. Y. C. & H. R.	Grade crossing. Fence with gates either side of tracks						Same as Spencer plan			Same as Spencer plan			Grade	L 2.[illegible]	L 2.5	Over	R but little	L	Under	L	R	Over	R 6.20	L 10.47
William	N. Y. C. & H. R.																			Under	L	R	Under	L 11.36	R 4.01
William	Erie													Under	L		Same as Buchholz plan			Grade			Under	L 15.57	

W. Spencer was ordered to Buffalo to make surveys, devise a plan of relief, and prepare an estimate of cost of same, giving due consideration to all interests involved.

Mr. Spencer's report was duly transmitted to the Board, the recommendations of which are shown in Figure 2.

Copies of this report were sent to the railroads and other interested parties, and after one postponement at the request of the railroads, a meeting was called by the Board of Railroad Commissioners to be held in Buffalo on the 19th of October, 1887.

At this meeting all interests were well represented, the railroads by their attorneys and engineers, and the City by its Engineer, Mr. Albert Krause, and by George E. Mann, C.E.

Mr. A. M. Wellington had previously been retained by a Committee of the Merchant's Exchange, and was expected to be present to submit a report, but on account of illness, was obliged to remain in New York. The report which he sent arrived less than an hour after the close of the hearing.

At this hearing the railroads offered a plan through their engineers: Mr. Katte, Chief Engineer of the New York Central and Hudson River Company, Mr. Buchholz, Engineer of bridges, etc., of the New York Lake Erie and Western Railroad Company, Mr. Handy, Chief Engineer of the Lake Shore and Michigan Southern Railroad Company, and Mr. McCreary, Engineer of Maintenance of Way of the Buffalo, New York and Philadelphia Railroad Company. (See Figure 2).

At the same time the counsel of one of the railroads threw out hints of another plan which was being prepared, but declined to speak farther in explanation of the matter. The plan referred to was doubtless the Buchholz plan, so called, which appeared later.

The report of Mr. Wellington, Mr. Krause and Mr. Mann followed closely after this meeting, and as they agreed in all essential features, they are classified as one report in Figure. 2.

Mr. Wellington in his report says:

"If we have a crossing of a single street and railroad in a flat country, it is a matter of indifference whether we raise the tracks and drop the street, or vice versa. If the track be underneath, it requires a nineteen foot clearance headway and the street only eleven feet, that is all. But if we have a network of tracks to separate from a network of streets, even partially, the case is different. It is impossible to raise the track at one place and the street at another indifferently; either the street must everywhere go underneath or everywhere on top.

Otherwise if the tracks be raised at one point and lowered at another there is a considerable stretch in between where a proper crossing of any kind is made impossible for lack of head room. There appears no room for doubt that the natural method in Buffalo is to raise the tracks and not the streets for several reasons . . . . . unless in a small district at the extreme south of the City where the conditions are special . . . . at the Ohio Basin which is the exceptional district referred to.''

The amount of railroad trackage in the City at this time was given by the City Engineer as about 450 miles, and the amount of traffic at Clinton Street—Erie Railroad—was said to be by actual count as follows:

In a day of eleven and a half hours, 152 trains and 708 teams, and the switching of trains occupied three full hours, when the street was absolutely obstructed.

The report of the Railroad Commissioners followed shortly after this, dated Nov. 10, 1887. (See Figure 2).

The following appears in the report:

''In a general way, however, it may be said that the tracks in that portion of the City south of the Hamburg Canal and west of Alabama Street are so closely connected with manufacturing and industrial establishments, elevators, etc., etc., that to elevate them would so seriously interfere with the industries of Buffalo as to be inexpedient, if not impossible.

To the north and west of this section the elevation of the entire system at present certainly would be exceedingly expensive, and probably undersirable. The Board deems, however, that in the future such elevation will not only be desirable but absolutely neccessary, and any plan for present relief which does not take this into consideration would be very unwise.''

Some of the difficulties of the problem were evidently well understood by the Board, as the following shows:

''Elk Street and Abbott Road; after a careful inspection of these two streets, while the Board is impressed with the great desirability of relieving them of the railroad obstructions at the various crossings, it is equally or more impressed with the difficulties of so doing. It cannot consistently recommend overhead highway bridges, for it believes that the tracks will eventually be elevated sufficiently to permit under crossings, nor can it recommend a depression of these two streets for the drainage will not admit of it.''

Clinton Street is mentioned as being similar to Elk Street and Abbott Road.

In Jan, 1888, a plan was prepared by Mr. Chas. W. Buchholz, Engineer for the Erie Railroad, the plan so mysteriously referred to by the railroad representative at the hearing of October 19, 1887.

Whether or not the plan was authorized by the Erie Railroad was not clear ; it was prepared with much detail, and doubtless represented Mr. Buchholz's best judgment rather than the plan submitted by the railroad engineers in October.

The plan (See Figure 2) provided for an exchange of lands between different railroad companies, that would simplify the problem and allow the railroads to enter a union terminal. Briefly the plan was as follows :

A Union Depot and Storage Tracks occupying the tract bounded by the Hamburg Canal, Exchange, Washington and Michigan Streets. Round House, Repair Shop and Yards bounded by the Canal, Exchange, Michigan and Louisiana Streets ; and also a Round House east of Louisiana Street.

A new street, (Warren Street), to be located about 370′ south of the Canal, and extending from Michigan to Alabama Street.

Scott Street to be discontinued.

Chicago Street to be discontinued from Warren to Seneca.

Alabama Street to be discontinued from Scott to Exchange.

Carroll Street widened from Michigan to Alabama and freight houses to be located on the two blocks bounded by Exchange, Carroll, Michigan and Alabama Streets.

Viaducts to be built on Michigan and Louisiana Streets, from Seneca to Warren.

Michigan street to be widened on the east side, from Exchange to Seneca, and on the west side from Carroll to Seneca.

Louisiana Street to be widened on the west side from Exchange to Seneca.

The plan was looked upon favorably, although some objected to the proposed viaducts at Michigan and Louisiana Streets.

On March 13th, 1888, a report was received by the Committee on Grade Crossings of the Merchants' Exchange, signed by the following gentlemen :

John Bogart, A. M. Wellington, George E. Mann, George W. McNulty, Walter Katte, C. W. Buchholz, Henry Flad.

This report was prepared in compliance to a request in the form of a resolution previously passed by the Grade Crossing Committee, "to consider and report upon a plan for the consideration of the committee."

The report is a modification of the Buchholz plan, and includes the union terminal feature. The plan also provides for admission of the Lehigh Valley Railroad by changing its line to the north side of the canal.

It is shown in detail in Figure 2.

All of the Thirteenth Ward south of Elk Street is mentioned as being so low and so largely given up to railroad interests that, "it seemed sufficient to provide a few main arteries of travel across it without attempting to provide for a normal development of City streets within it. Therefore, street crossings over the railroads with sufficiently easy grades seem unobjectionable at any point within this district where they are found more convenient."

For the territory north and east of the "Exchange street region," the Board advised that plans be prepared as soon as possible for the elimination of all grade crossings. The actual construction to be postponed until needed.

The Board recommended that this separation of grades be accomplished in some cases by raising the tracks, in others by depressing the streets, and in still others by changing the grades of both in part.

The report of the Board of Engineers was adopted by the Grade Crossing Committee, and a bill at once sent to the Legislature, which, as signed by the Governor on May 22, 1888, provided for the following as members of the Grade Crossing Commission of the City of Buffalo, any six of whom were authorized to enter into contract with the railroads on behalf of the City :

Robert B. Adam, George Sandrock, Edward H. Butler, Frederick Kendall, John B. Weber, Solomon Scheu, William J. Morgan, Charles A. Sweet, James E. Nunan.

As organized Mr. Adam was made Chairman, Mr. Morgan, Secretary, and Mr. Spencer Clinton was appointed Attorney.

Mr. Mann served in the capacity of Engineer to the Commissioners, originally as City Engineer, and, later in the employ of the Commissioners.

Later Mr. H. D. Kirkover was added to the Commission and Messrs. James Ryan and Augustus F. Scheu took the places of James E. Nunan, and Solomon Scheu, both deceased.

In October, 1897, Mr. Mann died, and Mr. Edward B. Guthrie was appointed to fill his place.

Aside from the changes noted, the Commission is at present, the same as originally organized. The members serve without compensation. They have been most painstaking and have at all times, enjoyed the complete and perfect respect of the citizens of Buffalo.

Immediately after organization the Commissioners began communicating with the various railroads for the purpose of entering into contract on the basis of the report of Mr. Bogart and others, which was met on the part of the railroads with delays and postponements.

Finally Mr. Depew of the N. Y. C. & H. R. R. R. openly declared that the Railroad Company he represented was unwilling to enter into any Union Terminal scheme, because of the greater facilities given other roads at the expense of his own, but with that feature cut out, the Central Railroad was willing to go ahead.

The Erie Railroad, through its President, Mr. King, disclaimed any knowledge of the Buchholz plan or of any Union Terminal scheme.

The plan of the Board of Experts was without doubt the most practical one that had been prepared, and it is to be regretted that conflicting interests prevented its being carried out.

In the latter part of 1888, the Lehigh Valley R. R. entered into a contract with the City, which, however, called for but a small change in the grades of the tracks, and that change incident to those in connection with the N. Y. C. & H. R. R. R.

During the seven years following 1888, the Commissioners met obstacles on every hand, and all that was accomplished was due to their grit and determination.

The N. Y. C. & H. R. R. R. signed a contract with the Commissioners on October 11, 1889, and supplemental contracts in March, 1891 and November 30, 1894.

The law as originally passed in 1888 was amended in 1890, and again in 1892, each successive act giving more power to the Commissioners, and receiving more opposition from the Railroads.

The Commissioners prepared a general plan previous to 1892, which in the number and location of structures agreed very closely with the one shown in Figure 1, but differing from it in the following respects :

The plan provided for taking land at the sides of approaches, by which means access to abutting property was maintained at the original ground elevation, which under the present plan is done in only a few cases, as at Michigan street, and the roadway for a 66′ street was but 32′ wide, instead of 42′ as under the present plan.

No sidewalks were provided on approaches to viaducts, but stairways instead, on either side of the railroad property, and sidewalks on the bridges immediately over the tracks.

At the hearings attendant upon this plan a well organized opposition was in evidence on the part of the residents south of Seneca street.

These opponents to the plan proposed were, no doubt, sincere in their opposition, but in many instances the opposition came from a lack of knowledge of the conditions which would result from the execution of the plan.

In July, 1892, Messrs. E. L. Corthell, of Chicago, and Augustus W. Locke, of North Adams, Mass., were invited by the Chairman of the Commissioners, pursuant to a resolution passed by the Common Council, to study the conditions existing, and also the plan proposed, and make a report, and to outline a plan which seemed most feasible.

The Chairman of the Board of Public Works was requested to act with the experts. Their report was presented in September, 1892. A vast amount of data was collected at that time, a part of which is given in Figure 3. The trackage shown in Figure 1 is taken from a map of that period.

FIG. 3.—TRAFFIC AT CROSSINGS IN 1892.—BUFFALO GRADE CROSSING IMPROVEMENT.

STREET.	TIME.	Pedestrians.	People in Vehicles.	Street Car Passengers.	Total Number of People passing along Street.	Number of Teams.	Number of Street Cars.	Number of Trains, Light Engines and Trains Switched.
Main and Terrace,	6 a.m.—6 p.m., July 20, '92	6991	3115	6772	16878	2140	575	153
	6 p.m.—6. a.m., July 20–1, '92	3496	520	3686	7702	436	210	36
	Total,	10487	3635	10458	24580	2576	785	189
Michigan	6 a.m.—6 p.m., July 20, '92	5340	2899	None	8239	2158	None	760
	6 p.m.—6 a.m., July 20–1, '92	1550	1162	"	2712	1149	"	128
	Total,	6890	4061	"	10951	3307	"	888
	† 5.30 a.m.–6.30 a.m., July 26, '92	1640	519	"	2159	397	"	53
Chicago	6 a.m.—6 p.m., July 28, '92	3285	597	None	3882	476	None	718
	6 p.m.—6 a.m., July 28–9, '92	650	61	"	711	39	"	508
	Total,	3935	658	"	4593	515	"	1226
	† 8 a.m.—9 a.m., July 28, '92	360	114	"	474	83	"	89
Louisiana,	6 a.m.—6. p.m., July 20, '92	885	1398	None	2283	1293	None	499
	6 p.m.—6 a.m., July 20–1, '92	305	410	"	715	344	"	118
	Total,	1190	1808	"	2998	1637	"	617
	† 10 a.m.—11 a.m., July 26, '92	425	294	"	719	254	"	67

† Hour of Maximum Travel.

Messrs. Corthell and Locke state in their report that their plan embraces the best features of the two plans, that of Mr. Spencer and the Buchholz plan.

It provided for joint elevated terminal passenger and freight stations, together with the general elevation of tracks. See Figure 2.

Engineer Mann in speaking of the plans, said, "They are utterly impracticable. Theoretically they are perfect, but it is impossible to carry them into effect. Everything is against them. It is a settled question, that there can be no union terminals, and without that, the plans are useless."

Gen. George S. Field, the Chairman of the Board of Public Works, submitted a minority report.

In that report he expressed practically the same ideas as those of Mr. Mann mentioned above, and concluded by recommending the plan of the Commissioners with certain changes as follows:—

Greater headroom over rails, generally 20 feet, but where freight cars are moved slowly, or over passenger tracks, "this could properly be reduced". Grades on approaches 4% or less. Approaches full width of streets.

These suggestions were followed in the main, and the general plan as modified, and as it exists today is that shown in Figure 1.

The whole question was again where it had been, which required dealing with the various railroads separately.

One after another the contracts were executed, the general conditions of which are shown in Figure 4.

For the sake of reference outlines of the State laws of New York, Massachusetts and Connecticut are given in this connection.

In the contract with the D. L. & W. R. R., the apportionment of cost was not fixed, but as provided in the Grade Crossing Act, an application was made to the Court for the appointment of a Commission of three to fix the apportionment.

Three non-residents of Buffalo were named and reported as shown in Figure 4.

The City of Buffalo is required by contract with the railroads to maintain both the Viaducts and Subways, together with the approaches.

Provision is made in the contracts with the Erie Railroad, the N. Y. C. & St. L. R. R. and the W. N. Y. & P. R. R. for the City to advance the money necessary for their share of the work as it is needed. The City raises all money for the improvement by issuing bonds, and the amount chargeable to the railroads mentioned is to be repaid to the City in 20 equal annual installments with interest at the same net

FIG. 4.—RAILROAD CONTRACTS WITH THE CITY.

Also conditions governing work done under State Laws and in the City of Chicago.—Buffalo Grade Crossing Improvement.

RAILROADS.	Date of Contract.	Clearance over top of rails.	City's proport'n of cost of total work.	City's proport'n of cost of lands taken.	City's proport'n of cost of consequential damages.	MAINTENANCE.	REMARKS.
N.Y.C. & H.R.R.R. and W.S.R.R.	Oct. 11, 1889. March, 1891. Nov. 30, 1894.	15′	⅓ of Approaches	⅓	½	Subways to carry streets under Railroads, Viaducts, and Bridges to carry streets over Railroad, and all approaches maintained by city.	
D.L. & W.R.R.	Mar. 6, 1895.	No Viaducts	45%	None taken.		Same as N.Y.C. & H.R.R.R. Contract.	
L.S & M.S.R.R.	Oct. 1, 1895.	16′	⅓ of Approaches	⅓	½	Same as N.Y.C. & H.R.R.R. Contract.	
Erie Railroad.	Feb. 8, 1896.	16½′	⅓ of Approaches	⅓	½	Same as N.Y.C. & H.R.R.R. Contract.	
B.C.R.R.	Feb. 8, 1896.	18′	⅓ of Approaches	None taken.	½	Same as N.Y.C. & H.R.R.R. Contract.	
N.Y.C. & St.L.R.R.	June 17, 1896.	18′	⅓ of Approaches	None taken.	½	Same as N.Y.C. & H.R.R.R. Contract.	
W.N.Y. & P.R.R.	Dec. 10, 1896.	16½′	⅓ of Approaches	⅓	½	Same as N.Y.C. & H.R.R.R. Contract.	
New York State Law.		Fixed by Board of R.R. Commissioners.	New railroad constructed across existing highway: Entire cost by Railroad. New highway constructed across existing railroad: Railroad ½, Municipality ½. Change in existing crossing to an overhead or an under crossing: Railroad ½, Municipality ¼, State ¼.			Abutments, railroad bridges and the framework of highway bridges, by Railroads. Subways, the roadway on highway bridges, and all approaches, by Municipalities.	State appropriates $100,000 each year for its share of work, or that amount reduced by what remains unexpended from the previous year.
Massachusetts State Law.		18	Railroad 65%; Municipality 10% or less. State the remainder, but no part, however, where grade crossing has been created since passage of act (1890).			Same as New York State Law.	State appropriates $500,000 each year for its share of work. Total expenditure by State not to exceed $5,000,000.
Connecticut State Law.		18′	Entire cost of Railroad, when change is requested by Railroad Company. When change is requested by Municipality and highway existed before railroad, ¼ or less by Municipality and remainder by Railroad; if Railroad existed before highway, Municipality ½ or less and remainder by Railroad.			Highway bridges maintained by railroads, as well as approaches to same when made with plank surface.	Railroads required to apply for abolition of one grade crossing for every 60 miles of road operated, in the State, each year.
Chicago—Agreement between City and Railroads.			Railroads pay entire cost.		Whole cost.		

rate as is borne by the bonds. In case the bonds sell at a premium the railroads receive their proportionate benefit of such premium.

In each contract, each individual structure is taken up separately and described in a general way.

Skeleton plans of each structure were also prepared, and are a part of the contract. The dividing lines between "approach work" and that paid for by the railroads alone are in general the exterior property lines of the railroads extended across the street. Everything between these lines is paid for by the several railroads interested in proportion to their frontage.

On the 22nd of May, 1895, the first shovelful of earth on the work at Michigan Street was turned by Mr. Mann, the Engineer for the Grade Crossing Commissioners.

Since that time the following pieces of work have been begun in the order given :

Michigan Street, Main Street and Terrace, Washington Street, Chicago Street, Clinton Street—Erie Railroad crossing, Seneca and Smith Streets, Swan and Seymour, William Street—N. Y. C. & H. R. R. R. crossing, Seneca Street—D. L. & W. R. R. crossing, and Seneca-Hamburg Streets.

All of the above have been completed, except the last, which was begun in July of last year, and will be completed in about a year from that time.

The Grade Crossing Act provides that the work may be built under the direction of either the City or the Railroads.

The approach work on Michigan Street, Main Street and Terrace, and on Washington Street was advertised and built by the City ; the superstructures and those parts of the substructures paid for by the N. Y. C. & H. R. R. R. alone, were built by that Company. Fortunately the substructural contractor for the City was also the contractor for the railroad company on all of this work and fearing that if in the future the same methods were followed in the letting of the work two contractors might be thrown together on the same ground, the Commissioners decided to either advertise the whole work on any crossing, or place it in the hands of a railroad company.

Accordingly Chicago Street, Clinton Street and Seneca-Smith Streets were advertised and built by the City ; Swan & Seymour and William Streets by the N. Y. C. & H. R. R. R. Company, Seneca Street by the D. L. & W. R. R. Company, and Seneca-Hamburg is being built by the City.

Plans for the substructure are prepared either by the City or by the railroad company having the work in charge, and are approved by all concerned.

The work when done by the City is advertised, and when let by the railroad company, bids are invited, and in either case the work is given to the lowest responsible bidder.

The superstructural work is under a separate contract, and is let in the same manner.

The bridge plans for Chicago Street and Seneca and Smith Streets were prepared by Mr. Albert Lucius, of New York, in the employ of the City, and those for Seneca-Hamburg and some other structures still to be built by Mr. F. E. Osborn of Cleveland, who was also employed by the City. All other superstructures were designed by the railroads building the same.

The term Viaduct is used in speaking of a crossing at which the street is carried by a bridge, or by bridges over the railroad, and the term Subway, where the street passes under the railroad. Except in special cases all that part of an approach to a viaduct underneath which a clearance of 13 or more feet can be attained, is built of steel, and the remainder of the approach, is made by building an abutment and retaining walls, and filling between with earth. (See Figure 6.)

The superstructure of a viaduct is supported either by piers or steel columns. Generally speaking the faces of abutments to subways are located at the street lines, and in some cases intermediate columns are placed in the street. The wing walls are never carried beyond the railroad property lines, and the earth in front of private property is excavated to a plumb line ; all on the supposition that eventually private property will be graded to the new street grade.

The Subways are covered with a tight floor, which excludes all light and air from below, except that coming from either end. This results in a rather damp and dark condition, which is certainly objectionable. It may be remarked in passing, that this feature is not unnoticed by the public, while the viaducts are spoken of favorably.

SUBSTRUCTURES.

The substructural work is advertised by the City under the several heads noted in Figure 5 ; prices are required for each and every item, whether a quantity appears for that item or not ; the prices include the labor of putting the material in place.

FIG. 5.—FORM OF BID SHEET FOR SUBSTRUCTURAL WORK—BUFFALO GRADE CROSSING IMPROVEMENT.

The following is a statement based upon approximate estimates made by the Engineer of the Commission, of the nature and quantities of the work and materials embraced in and required to fill the specifications, and which, while it is compiled with as much accuracy as possible in advance, must be understood as being approximate only, and the contractor must satisfy himself and verify its accuracy by personal examination of the site of the work and its surrounding conditions, and will not be allowed, at any time after the execution of a contract, to dispute the accuracy of this estimate, or set up any claims whatever based on its alleged inaccuracy, or to claim any misunderstanding in regard to the nature or amount of the work to be done or quantities of material to be furnished under such contract. The list of quantities hereto shall be attached and become a part of the contract.

QUANTITIES.

PLANS—1 to 6. STREET—ELK-ABBOTT.

DESCRIPTION.	QUANTITIES.	PRICE.
Brick.		
Brick, per M.	M.	14.00
Concrete.		
Concrete in foundation, etc., natural cement	285 cu. yds.	3.46
Concrete in sidewalks and about curbing, natural cement	410 "	3.40
Concrete, Portland cement	3,569 "	4.40
Crosswalks—Stone.		
Crosswalks to be taken up, including sand or concrete	828 sq. ft.	.02
Crosswalks to be relaid, including concrete and sand	"	.24
Crosswalks to be relaid, including sand	"	.13
Crosswalks, new, including concrete and sand	3,357 "	.36½
Crosswalks, new, including sand	"	.31
Curbing.		
Curbing to be taken up, except within lines of pits or trenches	2,568 lin. ft.	.02
Curbing, reset	699 "	.10
Curbing, new work, exclusive of concrete	4,500 "	.33½
Excavation.		
Excavation, solid rock	cu. yds.	2.45
Excavation, loose rock	"	1.25
Excavation in foundation, earth, including sewers, paving, curbing and crosswalks within lines of pits or trenches	7,370 "	.31½
Excavation for new subgrade in streets, earth	1,174 "	.30
Fill.		
Filling approaches and ramps to subgrade (excess of excavation), measured as specified	17,245 "	.25½
Filling cellars, measured as specified	81 "	.26½
Iron.		
Wrought iron	3,376 pounds.	.04
Cast iron	"	.03½
4-in. standard cast-iron pipe, built through walls, 6 ft. below top, for service connections	385 lin. ft.	.32½

Manhole.s		
Manholes, new, 3 feet high	3 Manh'ls.	12.66
Manholes, " 4 "	3 "	15.20
Manholes, " 5 "	4 "	17.50
Manholes, " 6 "	"	19.00
Manholes, " 7 "	1 "	20.00
Manholes, " 8 "	"	21.00
Manholes, " 9 "	1 "	25.00
Manholes, " 10 to 12 feet high	1 "	31.50
Manholes, " 12 to 15 "	"	45.00
Manholes, " 15 to 18 "	"	49.50
Manholes, " 18 to 20 "	"	53.00
Manholes, rebuilt	69 lin. ft.	1.75
Masonry.		
10-in. coping, abutments and approaches, Medina sandstone	11,667 sq. ft.	.69½
12-in. coping, abutments	169 "	.75
16-in. coping, on piers	611 "	.87½
12-in. wheel guard	"	.98
Bridge seats or column bases	1,193 cu. ft.	.70
Fender posts	54 "	2.00
Steps, Medina sandstone	"	2.00
First-class masonry	357 cu. yds.	9.00
Second-class masonry, retaining walls and ramps, including concrete filling	6,074 "	5.52½
Third-class masonry, retaining walls, etc.	"	4.52½
Pavement.		
Pavement taken up, stone, including concrete base	sq. yds.	.40
Pavement taken up, stone, including sand base	"	.32½
Pavement taken up, stone alone	1,977 "	.10
Pavement taken up, asphalt, including concrete base	174 "	.22½
Pavement taken up, asphalt, exclusive of concrete, except within lines of pits or trenches	"	.27½
Pavement relaid, Medina block, including concrete base	"	2.25
Pavement relaid, stone, on a sand base	95 "	.50
Pavement, new, Medina block, including a concrete base, with maintenance guarantee of 10 years	8,050 "	2.60
Pavement, new, stone, including sand	"	2.00
Pavement, new, stone alone	"	1.87½
Pavement, new, asphalt, including binder and concrete base, with maintenance guarantee of 10 years	1,719 "	2.50
Pavement, new, asphalt, including binder	899 "	1.77½
Piling.		
Piling in foundation	9,026 lin. ft.	.24½
Piling in coffer-dams	"	.28½
Piling in false work	"	.28½
Railing.		
Iron railing, including Posts shown on plan	3,295 "	1.92½
Cast-iron railing Posts, in addition to those shown	20 Posts	9.17½

Receivers.

Receivers, new	10	Receiv's	35.00
Receivers, reset	5	"	15.00

Sewers and House Connections.
Including Excavation and back filling.

Sewers, new, 36-in. brick		lin. ft.	3.49
Sewers, " 24-in. "		"	2.81½
Sewers, " 18-in. tile		"	1.90
Sewers, " 15-in. "		"	1.52½
Sewers, " 12-in. "	25	"	1.04
Sewers, " 10-in. "	1,630	"	.72½
Sewers, " 9-in. "	176	"	.60
Sewers, " 8-in. "		"	.62½
Sewers, " 6-in. "	838	"	.50
Sewers, relaid, 18-in. "		"	1.44
Sewers, " 15-in. "		"	1.25
Sewers, " 12-in. "		"	1.00
Sewers, " 10-in. "		"	.82½
Sewers, " 9-in. "		"	.68½
Sewers, " 8-in. "		"	.50
Sewers, " 6-in. "		"	.45

Sidewalks.

Sidewalks to be taken up, wood, except within lines of pits or trenches	7,830	sq. ft.	.01
Sidewalks to be taken up, stone, including sand, except within lines of pits or trenches	24	"	.02
Sidewalks relaid, wood		"	.10
Sidewalks relaid, stone, including sand		"	.12½
Sidewalks, new, stone, exclusive of concrete, but including two inches of sand	27,008	"	.22½
Sidewalks, new stone, including six inches of sand	8,652	"	.25

Timber.

Oak, per M.	Including spikes.	55.176	M.	40.00
Pine, per M., sidewalks, etc.			"	30.00
Hemlock, per M.			"	16.00
Beach or maple, per M.			"	24.50
Total				

NOTE.—Prices must be given for following items, which will govern in case Maintenance Guarantee is not required.

Pavement, new, asphalt, including binder and concrete base, without maintenance guarantee for 10 years	1,719	sq. yds.	2.46½
Pavement, new, Medina block, including a concrete base, without maintenance guarantee of 10 years	8,050	"	2.41

A sum sufficient for each inspector's pay of $5.00 per day is understood to be included in prices given.

A price must be given for each and every item in this list, whether a quantity appears or not.

NOTE.—The above prices were derived by taking an average of the middle two, or the middle price, of the ten under "Bid No. 1," received April 11, 1899.

These prices are a fair sample of those paid for completed work.

Bid No. 2 differed from Bid No. 1 in only three of the ten proposals, and in those three, was lower—4/10 of 1%, 9/10 of 1%, and 1 1/2%.

Recently the question has been raised by the Railroad Corporations; whether or not the improvement comes under the head of Municipal work, and is therefore subject to the City Ordinances and State Laws regarding the same.

The right to require a guarantee on the pavement has also been questioned by the railroads, and the points at issue are now in the Courts for an opinion, (April, 1899).

Pending this decision bids were received April 11, 1899, for the Elk-Abbott work.

Two separate proposals were submitted by each bidder, as follows:

Bid No. 1, based on doing the work according to the specifications complete, which include certain sections relating to labor organizations, hours to constitute a day's work, wages, cutting stone within the state, etc., in other words, on the basis of Municipal work.

Bid No. 2, is based on the above sections being omitted.

Each of the bids provides two prices for pavement, one with a ten year guarantee, and one without the guarantee.

In describing the methods, plans, etc., more in detail, the order followed will be that shown in Figure 5, omitting such items in the description, as seem self explanatory.

The following is based upon the latest plans and specifications:

Brick.—Brick is paid for as such only where used for special work not covered by the other items.

Concrete.—Concrete is made in the proportions of one (1) of cement, two (2) of sand and five (5) of broken stone. It is used as shown in Figure 6, and also in the construction of piers and about steel columns where the latter are carried to rock.

Crosswalks.—Crosswalk stone are specified to be not less than three and one-half (3½) feet in length by six (6) inches thick, and are either eighteen (18) inches or two (2) feet wide.

The foundation is in every case the same as that of adjoining pavement. The end joints are made diagonal to the line of travel to prevent ruts wearing between the stone. Medina sandstone is the material used.

Curbing.—Curbing is specified to be of Medina sandstone, four inches by eighteen (4″ x 18″) inches, and each stone not less than three and one-half (3½) feet in length.

On all approaches to viaducts, the curbing is set in concrete, as shown in Figure 6.

Where there is little or no fill, and therefore less danger of the curb settling from its true position, it is set in a full bed of sand, and backed with eight (8) inches of the same.

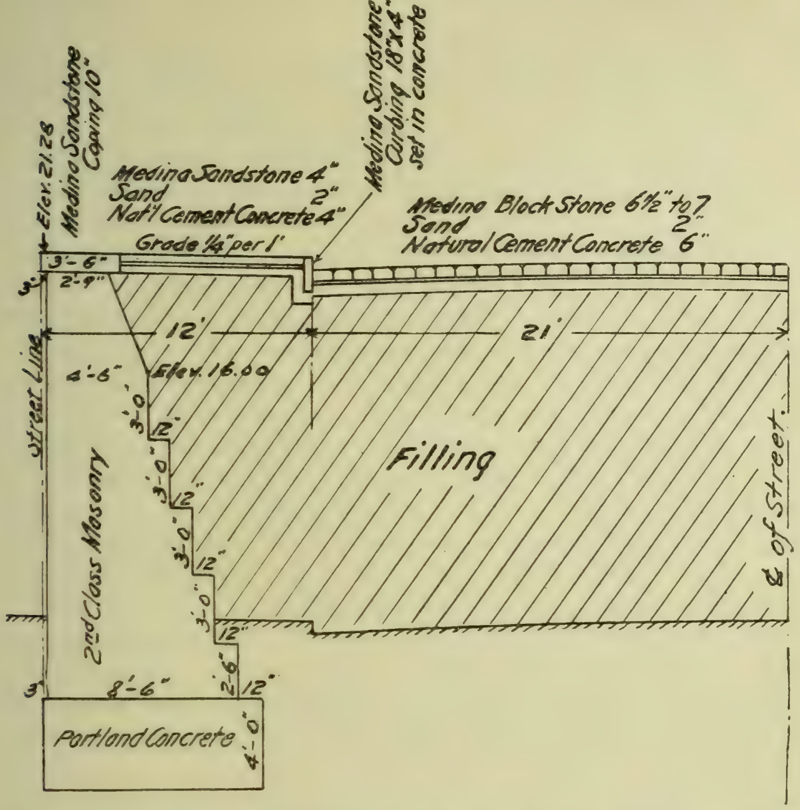

FIG. 6.—CROSS SECTION OF APPROACH TO ELK-ABBOTT VIADUCT AT WEST ABUTMENT. BUFFALO GRADE CROSSING IMPROVEMENT.

Fill.—As noted in Figure 5, only the fill in excess of excavation, is paid for as such; in other words, earth excavated is paid for but once.

The material available for fill in and about Buffalo, is clay, sand and loam, in varying proportions. It has been found necessary to allow the material to settle over winter before paving, to secure satisfactory results.

Michigan Street south approach, Washington Street and the approach in Exchange Street from the Chicago Street viaduct, were all paved shortly after the fill was put in place, and in each instance a settlement is apparent.

The pavement on Washington Street, and on the south approach of Michigan Street, is the stone from the old pavement relaid on a sand base, while the fill was nearly all sand.

On Exchange Street the pavement is block stone laid as shown in Figure 6.

The extreme fill on Washington Street and on Exchange Street is about six and one-half (6½) feet, and on Michigan Street only two (2) feet.

On other pieces of work where the extreme depth of fill, in some cases is as much as sixteen (16) feet, and the pavement a block stone on concrete, put in place after a winter's settlement, the results have been very satisfactory, and but little, if any settlement is apparent.

When the opening of a viaduct to travel will be seriously delayed by waiting for the pavement, temporary plank roadways are built for use until such time as it will be safe to lay the pavement.

The fill is deposited by wagon load, or thrown up by shovelful from the trenches on either side where the retaining wall is to be built. The specifications require that the material shall be levelled in layers not exceeding six (6) inches in depth, (previously 12 inches) and rolled or pounded. The earth is kept sufficiently moist to compact well. The great amount of teaming over the fill, and the piling of stone upon it during the progress of the work, undoubtedly aid in compacting the material.

The whole question of filling, the material used, etc., is one of the most important in the entire work, and the maintenance guarantee on the pavement places the contractor in a position to do the work of filling in the best manner possible, that the cost of paving repairs will be a minimum.

Iron.—Wrought iron is used principally in the shape of twenty (20) inch by three-fourths (¾) inch drift bolts in the grillage for piers.

Cast iron is little used.

Manholes.—Manholes to sewers are built of brick, with eight (8) inch walls, and are from two (2) feet (at the top) to four (4) feet (at the bottom) in diameter.

Cast iron heads 12 inches deep, and cast iron perforated covers complete the top of the manholes.

Masonry:—Coping.—Medina sandstone coping is specified for the tops of retaining walls and abutments of viaducts, for the reason that its use is in part that of a sidewalk. Yammerthal stone, the local stone specified for first and second class masonry, while excellent for the purpose for which it is used, is unsuitable for sidewalks because it becomes smooth with wear.

The top and face of the coping is finely bush-hammered. The 12 inch coping on abutments is laid at the same elevation as the bridge seats, and extends from the seat to the end of the abutment; it is merely for a finish and the same stone is used as for the abutment proper. It is bush-hammered on top and has a chisel draft on all exposed edges. The 16 inch coping, the bridge seats and the column bases are in all respects similar to the 12 inch coping and differ only in dimensions and in the uses to which they are put.

Wheel Guard.—The wheel guard is used on approaches, such as that on Exchange Street, where there are no sidewalks; it is of the same stone as that used for the retaining walls and is surmounted by a 10 inch coping of the same stone. The top and inner corner is rounded to a 6 inch radius, and serves as a curb to the pavement.

Fender Posts.—Fender posts are made from Medina sandstone, and are set at the abutments as a protection to the end of the iron work. They are 18 inches square, and stand about 2 feet above the pavement.

First Class Masonry.—First class masonry is used in piers from below the ground surface to the coping, and is specified to be of Yammerthal stone, or of stone equally good, approved by the Engineer.

A light cream colored sandstone, from near Oil City, Pa., has been used on much of the work. It stood a Government crushing test nearly equal to that of granite, and hardens on exposure. It is a very satisfactory stone.

The Yammerthal stone previously mentioned, is a flinty limestone, and although somewhat difficult to dress, is in every way satisfactory.

The stones are cut to the same dimensions throughout appearing on the face, and are laid with one half (½) inch joints.

The piers for an average span of 125 feet are 5′–6″ under coping in a direction longitudinally of the bridge, and increase toward the ground with a ½ inch per foot batter on either face. The other dimension is 7′–6″ under coping and increases with a one inch per foot batter.

The corners have a one inch chisel draft on either face.

The courses vary in thickness from 14 inches to 24 inches, and average about 18 inches.

Second Class Masonry.—Second class masonry is used in abutments, piers and retaining walls, any one course of which is the same thickness throughout, and is laid with ½ inch joints. The stone used is the same as that in first class work, and while 12 inches is specified as the minimum thickness of stones in any course, the same average on work done holds good that applies to first class work. One-third of the entire work is headers. The backing is of roughly squared stone, at least 2 cubic feet in size, and all spaces or voids are filled with concrete.

Originally this was not done, but spawls were used instead; under that method there was always danger of a lack of mortar at the right place, especially when the mason work was sublet by the yard, as it was in some instances.

Second class work includes the entire masonry in the abutments and retaining walls, with the exception of coping, etc, noted above, and the concrete footing, (See Figure 6). Retaining walls to viaducts are designed such that the thickness at any point above the concrete footing is at least $\frac{3}{7}$ths of the height above that point, (for abutments $\frac{3}{7}$ to $\frac{1}{2}$). This has been found ample for the fill described above. The east abutment of the New York Central railroad bridge over State Street, Rochester, N. Y., was built of a width at the ground about 0.40 of the height to top of ties and failed by separating from the retaining or wing walls.

The filling back of the abutment was a soft red sand.

Third Class Masonry.—Third class masonry is but little used, and only on work of minor importance.

The specifications provide for laying masonry in freezing weather when the thermometer is not lower than 22 degrees F., by using hydraulic lime in mixture with the cement mortar and tempering with brine.

Pavement.—Block stone pavement is used on approaches of 4% ± ; and in Subways.

The material used is the best quality Hard Medina Sandstone, so cut that the stones will lay with joints not to exceed ½ inch.

The stones are bedded in a sand cushion over a concrete base, as shown in Figure 6. In all the work thus far done, the joints between the stones have been filled with a Portland Cement grout, made of one part of cement to one part of sand.

On the north approach of Michigan Street, the grout was poured from the mixing trough over the surface of the pavement and swept down the 4% incline until it disappeared. Considerable care was exercised in seeing that all the joints were filled, and but little head given to the surplus cement that might lie on the surface. This resulted in a pavement that has stood up remarkably well, but so smooth that, together with the grade on which it is laid, it has been found necessary to cut out the cement from the joints for a depth sufficient to give a footing for horses.

Since then all surplus cement has been removed from the surface of the pavement, when it is laid. This is done either with wire brooms or by means of a stream of water from a hose held close to and parallel to the surface.

Asphalt pavement is used only on viaducts and other places where the grades are comparatively light, although there are streets in Buffalo paved with asphalt having grades of 5.10%, 4.40%, 3.90%, etc. The foundation for the pavement is the same as for block stone 6 inches of concrete ; over this is laid a 1½ inch course of binder and 2 inches of "topping," or wearing surface.

In bids recently received, the difference in prices for pavement with or without a 10 year guarantee were almost uniformly 30 cents per yard for asphalt and 15 cents per yard for block stone. These figures doubtless represent the actual cost of maintenance rather than those in Figure 5.

Piling.—Piling is used principally under piers, where the rock is at too great a depth to build the masonry immediately upon it. The piles are of beech, maple or oak, about 7 inches in diameter at the small end, and 12 inches at the butt end, and are driven to rock, about 3 feet centers either way ; of sufficient number that the loading per pile is about 20 tons. The tops of piles are cut off at a level from 9½ to 10 feet below ground surface, and above this level are built successively 2 feet of grillage, 4 feet of Portland Cement concrete, and following this the second class masonry.

Before the piles are driven the ground is excavated to a depth about 2½ feet below the proposed elevation of tops of piles, and in the course of driving, this distance is reduced, that is the ground raises about 18 inches, the amount varying with the number of piles driven, and with the nature of the soil.

The piles are sawed to the proper grade and the surrounding ground is leveled 1 foot below the tops of the piles, 12″ x 12″ hemlock cap timbers are then drift bolted to the piles and natural cement concrete filled in between the timbers and about the heads of the piles to the tops of the timbers.

A solid floor of 12″ x 12″ hemlock is then laid across the caps, and on this is built the Portland Cement concrete, as noted above.

Railing.—A hand-rail 3′6″ high is built on approaches to viaducts of the same design as that on the viaduct.

Receivers.—Receivers of local make, built of cast iron are set at necessary points at the line of the curb, and connected with the sewers by 9 inch tile.

Sewers.—The price per foot for sewers includes the cost of excavation and back fill.

Sidewalks.—The material for sidewalks is Medina sandstone. The stones are at least 2½ feet long, 2 feet wide and 4 inches thick, and on approaches to viaducts are laid as shown in Figure 6.

Where the fill beneath is slight, the stone are bedded in 6 inches of sand, without the concrete base.

Timber.—Timber is used as noted under piling.

SUPERSTRUCTURES.

All superstructures over subways are built with tight floors, either of troughs or of buckle-plate.

On the work of this character along the line of the N. Y. C. & H. R. R. R., girders supporting the floor have been erected along each curb line and along the center line of the street, and the flooring laid transversely to these lines. The troughs are filled with binder and ballast, and the ties and rails are laid as on adjoining ground.

By this method the Railroad Company is in a position to shift the tracks at any time it may be desirable, and still a shallow floor system is secured. As an example, on Swan and Seymour streets the troughs are 14 inches deep, ballast above troughs 2¾ inches, ties 6 inches and rails 5⅛ inches, a total of 2′3⅞″. The top and bottom plates are 13½″ x ½″, vertical plates 13″ x ⅜″, and angles 3″ x 3″ x ⅜″. This is for a span of about 22 feet. The binder is laid to a grade, pitching toward the curb girders, where there are openings through which the water passes to the gutter below.

The types of superstructures used on viaducts have been as follows :

A pin connected truss span ; deck lattice girders ; through plate girders ; deck plate girders, and through double intersection lattice truss spans. The last two types being used more than the others. (See Fig. 7).

Girders are used for spans varying from 40′ to 80 or 85′, and for greater spans trusses are used ; the pin connected truss bridge was built for a 194 foot span on Seneca Street.

Where the clearance beneath will allow of it, deck girders are used in preference to through girders, for the reason that the sidewalk is not separated from the roadway, but the two occupy the same relative position as on the masonry approaches, thus presenting a much better appearance and requiring less attention for maintenance than on through girder construction.

The floor system in some instances where the clearance below was not an important factor, has been made up of brick arches built between 9 inch or 10 inch I beams, which were spaced about 4 feet centers. The floor beams in such cases were spaced about 8 or 9 feet.

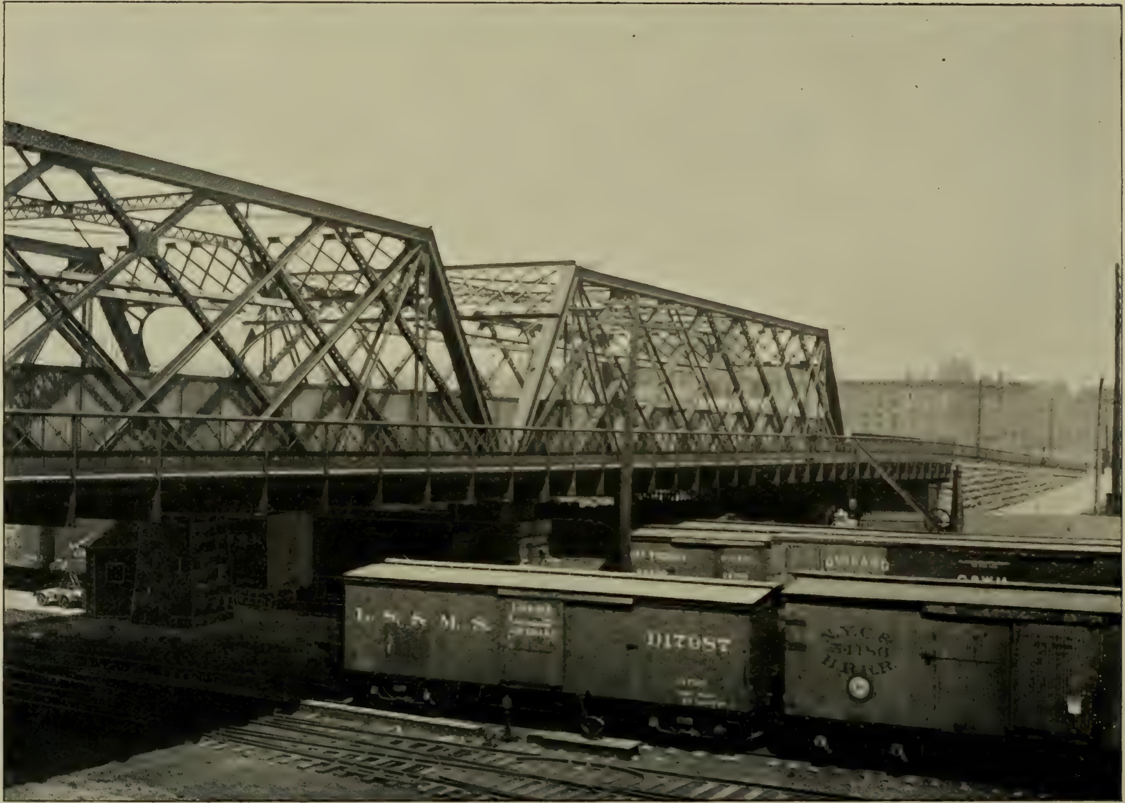

FIG. 7.—CHICAGO STREET LOOKING NORTH FROM EXCHANGE STREET—BUFFALO GRADE CROSSING IMPROVEMENT.

NOTE.—Tracks shown are between Exchange and Carroll Streets. They were laid since 1892, and therefore are not shown in Fig. 1. They lead to the new freight house of the N. Y. C. R. R., situated between Chicago and Michigan Streets.

FIG. 8.—COST OF STRUCTURES UNDER CONTRACT PLAN.—BUFFALO GRADE CROSSING IMPROVEMENT.

STREET.		CITY.	R.R'S.	TOTAL.	REMARKS.
†Franklin	Superstructure.	988	. . .	988	Not inclu. in contact with R.R.
†Pearl	"	. . .	2,038	2,038	
†Main and Terrace	Substructure.	7,081	79,784	86,865	Including one-half the work between Main and Washington Streets.
	Superstructure.	. . .	9,965	9,965	
	Consequential.	15,137	15,137	30,274	
†Washington	Substructure.	3,755	26,347	30,102	Including one-half the work between Main and Washington Streets.
	Superstructure.	. . .	7,000	7,000	
	Consequential.	—Not fixed.—			
†Michigan	Substructure.	12,627	30,053	42,680	
	Superstructure.	13,786	69,533	83,319	
	Land Awards.	58,862	117,723	176,585	
	Consequential.	50,717	50,717	101,435	
†Chicago	Substructure.	22,408	35,669	58,077	
	Superstructure.	25,560	93,388	118,948	
	Land Awards.	64,102	128,204	192,306	
	Consequential.	—Not fixed.—			
†Swan and Seymour	Substructure.	6,850	49,870	56,720	
	Superstructure.	. . .	39,950	39,950	
	Land Awards.	10,776	21,387	32,163	
	Consequential.	—Not fixed.—			
†William—N.Y.C.R.R.	Substructure.	7,323	48,899	56,222	
	Superstructure.	. . .	31,416	31,416	
	Land Awards.	7,405	14,810	22,215	
	Consequential.	—Not fixed.—			
†Clinton—Erie R.R.	Substructure.	18,687	75,950	94,637	
	Superstructure.	. . .	34,133	34,133	
	Land Awards.	9,410	18,819	28,229	
	Consequential.	—Not fixed.—			
†Seneca and Smith	Substructure.	43,737	94,437	138,175	Widening of Smith Street not included.
	Superstructure.	27,884	108,649	136,532	
	Consequential.	—Not fixed.—			
†Seneca—D.L. & W.	Substructure.	16,612	16,612	33,223	Includes raising R.R. tracks from W.N.Y. & P.R.R. to Elk St., also Bailey Av., a distance of 380 ft.
	Superstructure.	5,368	5,368	10,737	
	Consequential.	—None.—			
Abbott Road—B..C.R.R.	Substruct're and Superstructure taken together.	59,775	103,314	163,089	Cost estimated.
Abbott R'd and Elk St., D.L. & W.	"	44,181	53,998	98,179	" "
Babcock—W.N.Y. & P.	"	29,989	94,117	124,106	" "
Clinton—B.C.R.R.	"	7,659	30,836	38,495	" "
Clinton—N.Y.C.R.R.	"	4,254	75,454	79,708	" "
Eagle	"	4,573	76,355	80,928	" "
Elk-Abbott	"	100,776	264,975	365,751	" "
Elk—B C.R.R.	"	60,179	97,383	157,562	" "
Emslie	"	7,403	72,155	79,558	" "
Fillmore—N.Y.C.R.R.	"	5,112	85,232	90,344	" "
Fillmore—W.N.Y. & P.	"	12,445	54,357	66,802	" "
Howard	"	4,423	88,181	92,604	" "
Lord	"	3,458	50,406	53,864	" "
Louisiana— N.Y.C.R.R.	"	91,734	268,499	360,233	" "
North Division	"	4,292	92,662	96,954	" "
Oneida	"	4,435	74,477	78,912	" "
Perry—Erie R.R.	"	29,916	78,984	108,900	" "
Perry—L.S.R.R.	"	39,181	123,908	163,089	" "
‡Seneca and Hamburg	"	81,212	288,686	369,898	Cost based on contractor's bids, and is subject to slight change.
	Land Awards.	21,547	43,094	64,642	
Smith—N.Y.C.R.R.	Substruct're and Superstructure taken together.	4,090	49,883	53,973	Cost estimated.
South Division	"	4,289	100,665	104,954	" "
Van Rensselaer—E.R.R.	"	34,445	73,655	108,100	" "
William—Erie R.R.	"	15,049	38,700	53,749	" "

SUMMARY OF COST—NOT INCLUDING CONSEQUENTIAL DAMAGES.

	CITY.	RAILROADS.
Structures completed and under contract .	$ 293,878	$1,147,747
Lands taken	172,102	344,037
Structures yet to be built	571,658	2,048,196
TOTAL	$1,037,638	$3,539,980

† Structures completed. ‡ Structure under contract.

NOTE: The above amounts do not include the cost of raising or lowering the railroads, except along the line of the D. L. & W. R.R.

On Michigan Street a trough floor was used, the troughs running parallel to the floor beams, and resting on small I beams attached to the floor beams;—the tops of troughs on the same elevation as the tops of floor beams,—and by this method, with the trusses 42 feet centers, a floor depth of 2′–5″ was attained, (floor beam 1′–10½″, binder 4–½″, asphalt 2″).

Of course a concentrated load was not distributed beyond the adjoining floor beams by this method, and to overcome this, the troughs were run longitudinally to the street, in designing the structures for Chicago Street and Seneca-Smith Streets, which added the depth of trough (4″) to the depth of floor.

Stone block pavement is used on approach spans, where the grade is 4% ±, and asphalt on all spans where the grade is light.

The floor on the Seneca-Hamburg structure is inverted buckle plate on floor beams, which are spaced from 3′ to 4′ apart; the sidewalks are also of buckle plate covered with concrete and a cement wearing surface. (See *Engineering Record* for May 13, 1899). No wood is used in the construction of this structure proper, although it is used for the tread to steps and in the roofs of the several stairways leading to it, at Jefferson St., Exchange St., etc. Wooden ceilings are attached to the under side of floor-beams as noted below. The result of the blast from the locomotives, acting on the iron of the superstructure immediately above, has been found so dissastrous, that light wooden ceilings are now provided on all spans over railroad tracks. Various paints have been used, but without other protection, the results have in all cases been unsatisfactory.

On all work now complete the sidewalks are of 3 inch yellow pine laid on timber stringers. Figure 8 gives the cost of structures already built, and the estimated cost of those still to be built. The cost of construction, on completed work, has been only about 74% of the original estimate, and is due principally to the low prices which have prevailed. A similar difference, though perhaps, not so marked, may be looked for, on work still to be built.

The damages awarded property injured by the building of any structure, are fixed by a Commission of three (3), appointed by the court; one or more Commissions being appointed for each structure. These awards are confirmed by the court.

No time can be fixed for the completion of the work outlined by the General Plan, for the reason that delays, legal and otherwise, are constantly arising, and are a more potent factor than the work of construction. However, with the railroads and the Commissioners now working toward the same end, the way seems clear.

REFERENCES:

Article by Mr. Edward B. Guthrie, in the *Railroad Gazette* of May 6th, 1892.

Corthell and Locke report, Sept. 6, 1892.

Article by Mr. George E. Mann, read before the Buffalo Society of Natural Sciences, March 9, 1895.

TRANSACTIONS

OF THE

ASSOCIATION

OF

CIVIL ENGINEERS

OF

CORNELL UNIVERSITY

VOL. VIII, 1899-1900

CONTAINING

ADDRESSES BY NON-RESIDENT LECTURERS, MISCELLANEOUS
PAPERS, CONSTITUTION, AND LIST OF MEMBERS
OF THE ASSOCIATION

NOTE.—This Association is not responsible for any statements or opinions advanced in any of its publications.

ITHACA, NEW YORK
MAY, 1900.

PRESS OF
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TRANSACTIONS OF THE

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CORNELL UNIVERSITY.

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Copies of this and preceding issues can be obtained from Professor C. L. Crandall, Corresponding Secretary of the Association of Civil Engineers of Cornell University, Lincoln Hall, Ithaca, N. Y.

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CONSTITUTION

OF THE

ASSOCIATION OF CIVIL ENGINEERS

OF CORNELL UNIVERSITY.

PREAMBLE.

We, the undersigned, members of the Senior and Junior classes in the College of Civil Engineering of Cornell University, do hereby form ourselves into an Association for the discussion of engineering topics, and the promotion of general information on engineering subjects, and do hereby agree to abide by, and sustain the following Constitution and By-Laws :

ARTICLE I.

NAME.

1. This Association shall be known as the Association of Civil Engineers of Cornell University.

ARTICLE II.

MEMBERSHIP.

1. The Association shall consist of Active and Honorary members.

2. All Alumni of this college and all students recognized as upperclassmen, and registered in the College of Civil Engineering, are eligible to membership in this Association.

3. Any eligible person may become an honorary member by a two-thirds vote of the members present at any regular meeting. Such members shall have privileges of active members except those of voting and holding office, and shall be exempt from all dues.

4. The membership fees for this Association for all active graduate members shall be $1.00 per annum. All money received from such membership fees shall be devoted to defraying cost of publication of non-resident lectures delivered before the Association. All other expenses of this Association shall be met by direct tax upon the undergraduate members.

5. A copy of each lecture delivered before this Association shall be forwarded to each member of the Association.

ARTICLE III.

OFFICERS.

1. The officers of the Association shall consist of a President, Vice-President, Recording Secretary, Corresponding Secretary, and Treasurer.

2. The President shall preside at all meetings of the Association and enforce the Constitution and By-Laws, and shall call special meetings at the request of five active members.

3. The Vice-President shall take the chair at the request of the President, and shall act as President in his absence. The Vice-President shall be chairman of the appointment committee.

4. The Recording Secretary shall keep minutes of the proceedings of all meetings of the Association and shall post notices for the same.

5. The Corresponding Secretary shall attend to all the necessary correspondence of the Association. He shall be elected from the Faculty of the college.

6. The Treasurer shall receive all money and dues, and shall pay all bills of the Association, such bills to meet the approval of the Executive Committee before such payments. He shall make a report when called upon by the Association and also when his term of office expires. He shall be chairman of the Executive Committee.

7. The officers shall be chosen by ballot at the last regular meeting of the spring term, from the Junior Class, and shall hold office until their successors are elected.

8. All vacancies shall be filled by a special election—only members of the Senior Class being eligible to hold office.

ARTICLE IV.

COMMITTEES.

1. There shall be three Standing Committees, an Executive Committee, a Committee on Appointments, each consisting of three members, and a Publication Committee, consisting of five members. These committees shall be appointed by the President at the beginning of the year.

2. The Executive Committee shall see that the rooms of the Association are ready for occupancy previous to all meetings, and shall transact such business as may be referred to it by the Association.

3. The Committee on Appointments shall make appointments for all literary exercises for each meeting, and such appointments shall be posted at least two days before the meeting. The committee shall furnish the Secretary with a list of such appointments.

4. The Publication Committee shall have charge of the compiling, publication, and sale of the Transactions.

ARTICLE V.

AMENDMENTS.

The Constitution or By-Laws may be amended by a two-thirds vote of all members present at any regular meeting; such amendment to be before the Association at least one week.

AMENDMENT I.

The initiation fee for membership to this Society shall be one dollar, and this fee shall cover all dues for the first year of membership. For each succeeding year, an annual due of one dollar shall be charged. The annual membership fee shall entitle the member to a copy of the Transactions published during each year of membership.

BY-LAWS.

ARTICLE I.

REGULAR MEETINGS.

Regular meetings shall be held once in two weeks, in the Association rooms, commencing on the first Friday after registration week, and ending on the last Friday but one before examination week of each term.

ARTICLE II.

QUORUM.

One-third of the active undergraduate members of the Association shall constitute a quorum. No business can be transacted without a quorum being present.

ARTICLE III.

ORDER OF PROCEEDINGS AT A REGULAR MEETING.

1. Roll call.
2. Minutes of Preceding Meeting.
3. Literary Exercises.
4. Unfinished Business.
    - *a.* Report of Standing Committees.
    - *b.* Report of Special Committees.
    - *c.* Report of Officers.
    - *d.* Miscellaneous Business.
5. New Business.
6. Adjournment.

ARTICLE IV.

EXERCISES.

The exercises shall consist of discussions, memoirs, essays, papers, lectures, and such other exercises as the Association shall from time to time direct.

ARTICLE V.

SUSPENSION OF BY-LAWS.

A By-Law may be suspended for one meeting by a vote of two thirds of the members present.

H. R. LORDLY,
E. J. FORT,
H. D. ALEXANDER,
Committee.

OFFICERS FOR 1899-1900.

President,

LEON D. CONKLING.

Vice-President,

A. H. VON BAYER.

Treasurer,

C. W. COIT.

Corresponding Secretary,

PROESSOR CHARLES L. CRANDALL.

Recording Secretary,

GEORGE W. PENFIELD.

Appointment Committee,

A. H. VON BAYER, *Chairman,*
M. E. SHIRE,
H. O. EGEBERG.

Executive Committee,

C. W. COIT, *Chairman,*
W. E. CONKLIN,
W. J. CUNNINGHAM.

Publication Committee,

W. O. WHITE, *Chairman,*
J. A. VOGLESON,
J. H. MINER,
R. A. PENDERGRASS,
G. E. J. PISTOR.

PRESIDENT'S ADDRESS.

To the Members of the Association of
Civil Engineers of Cornell University,

GENTLEMEN :

One of the most successful and pleasant years in the history of our Association ends to-day. This is the last time that I shall have the honor and pleasure of speaking to you as your President ; an honor I have tried to deserve in both action and spirit.

I wish to thank my brother officers and the members of the Association for their kind and considerate treatment during my term of office.

The past year has seen a solid growth in interest and attendance at our bi-weekly meetings. We have been very fortunate in the number and excellence of the non-resident lecturers. The different members of the Faculty have with their usual kindness responded to our requests for lectures. On behalf of the Association I wish to express to the members of the Faculty, and others who have lectured before this Association, our sincere and hearty thanks.

The Transaction Board has worked long and faithfully upon a volume that does credit to itself, to the Association, and to the Civil Engineers of the Class of 1900.

We now turn over to the Class of 1901 the active management of the Association, and I earnestly advise the present Junior class to make a sincere effort to associate with, and to become well acquainted with the men into whose hands they in turn will entrust the Association. Make an early and systematic canvas of the Class of 1902, presenting to each man the idea and the object of the Association, and a cordial invitation to become an active member.

I would urge upon the Appointment Committee the absolute necessity of making a complete programme early in the year to insure the regularity of the meetings. If the members will prepare papers and have discussions upon them, I am sure they will derive much benefit.

The members of the Graduating Class of 1900, now bidding you farewell, take with them many pleasant memories of the hours spent as members of this Association. They will always be interested in the welfare of this Association and its members.

Gentlemen, I take much pleasure in presenting the chair to Mr. Wilgus, your President for 1901.

LEON D. CONKLING.

NON-RESIDENT LECTURERS,

1899-1900.

F. H. Newell, C.E., Hydrographer U. S. Geological Survey.
Hydrographic Investigations of the U. S. Geological Survey.

Major Thomas W. Symonds, Corps of Engineers, U. S. Army.
Canals from Lakes to Sea.

Forrest M. Towl, C.E., '86, M.Am.Soc.C.E., M.A.S.M.E., Engr. Nat. Transit Co.
Oil Wells.

George W. Tillson, C.E., Principal Assistant Eng. Dept. Highways.
Asphalt and Asphalt Pavements.

*S. Whinery, C.E., M.Am.Soc.C.E., Vice-Pres. and General Mgr. Warren Sharp Asphalt Paving Co.
Economics of Street Paving.

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* Published in this volume.

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UNDERGRADUATES.

Albert Lloyd Colsten,	1895
William Mackintosh,	1896
Gilbert Powers Ritter,	1897
Harley Stuart Gibb,	1898

GRADUATES.

John Fillmore Hayford,	1895
Elon Huntington Hooker,	1896
Erasmus Darwin Preston,	1897
John Cassam Wait,	1898
Frederic Eugene Turneaure,	1899

The Fuertes gold medals are of a value of fifty dollars each, and are awarded under the following conditions :

The undergraduate medal "will be awarded annually to that student of the College of Civil Engineering who on graduating may be found by the faculty of the college to have maintained the highest degree of scholarship in the subjects of his course ; provided he has been in attendance at this University for at least two years."

The graduate medal "will be awarded annually to that graduate of the College of Civil Engineering who may write a meritorious paper upon some engineering subject tending to advance the scientific or practical interest of the profession of the Civil Engineer."

The papers shall be presented on or before April 15. If a paper is submitted in printed form, it will be received only provided it has not been published earlier than the last preceding April 15.

OFFICERS FOR 1900-1901.

President,
H. S. WILGUS.

Vice-President,
C. L. WILCOX.

Treasurer,
B. H. BISBEE.

Corresponding Secretary,
PROFESSOR CHARLES L. CRANDALL.

Recording Secretary,
W. E. WILSON.

Appointment Committee,
C. L. WILCOX, *Chairman,*
R. B. WILLIAMS, JR.,
M. G. HILPERT.

Executive Committee,
R. F. PROCTOR, *Chairman,*
G. O. BEUCK,
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Publication Committee,
G. E. J. PISTOR, *Chairman,*
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ARTHUR ADAMS,
E. B. WHITMAN,
ROBERT FOLLANSBEE.

STUDENT MEMBERS OF THE

ASSOCIATION OF CIVIL ENGINEERS,

CORNELL UNIVERSITY.

GRADUATE.

G. G. Smith.

1900.

H. H. Bassett,
A. H. von Bayer,
W. R. Bright.
C. T. Chapman,
C. W. Coit,
W. E. Conklin,
L. D. Conkling,
W. J. Cunningham,
J. C. Davis,
H. O. Egeberg,
F. J. Engel,
G. I. Finley,
E. W. Gehring,
L. C. Giltner,
D. A. Ketchum,
C. W. Landis,
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R. A. Pendergrass,
G. W. Penfield,
S. Reyna,
B. W. Rosekrans,
M. E. Shire,
W. C. Spiker,
C. M. Stegner,
E. J. Strasburger,
H. W. Swanitz,
T. W. Taylor,
J. C. Trautwine, 3d,
J. A. Vogleson,
W. O. White,

1901.

A. F. Armstrong,
G. O. Beuck.
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C. B. Brown,
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C. L. Hartwell,
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G. E. J. Pistor,
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N. O. Tiffany, Jr.,
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E. B. Whitman,
C. L. Wilcox,
H. S. Wilgus,
H. S. Williams,
R. B. Williams, Jr.,
W. E. Wilson,
C. H. Zolzer,

MEMBERS OF THE

ASSOCIATION OF CIVIL ENGINEERS,

CORNELL UNIVERSITY.

ALL GRADUATES OF THE COLLEGE OF CIVIL ENGINEERING UP TO AND INCLUDING 1900.

The address on the line with the name is the home or permanent one; that on the line below is the business or usually more temporary one. When only one is given its position is an index of its permanency, except when forced from one position to the other by lack of room. All the addresses have been verified since March 1, 1900, except those in italics, which are unreliable. All changes in address should be reported promptly to the Corresponding Secretary. The names of the deceased members are given at the end of the regular list.

Adair, Arthur P. . . . *B.C.E.*, *C.E.*, '99 . . . . Moscow, Idaho.
Asst. Engr., B. & O. S. W. R.R., Osgood, Ind.
Agate, Elroy T. . . . . *C.E.*, '97 . . . . Pittsford, N. Y.
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Alexander, Fred B. . . . . . . *C.E.*, '74 . . . . . 210 Macon St., Brooklyn, N. Y.
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Ashby, Charles W. . . . . *C.E.*, '93 . . . . 11 Hudson Ave., Troy, N. Y.
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U. S. Dep. Surveyor, Rampart City, Alaska.
Avery, Fred H. . . . . *C.E.*, '97 . . . . 424 E. Superior St., Chicago, Ill.
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Ayres, C. Morton . . . . . . *C.E.*, '94 . . . . Tuscaloosa, Ala.
Inspr. Ch. Const. Lock No. 4, Black Warrior River, U. S. Engr's. Office.
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Bailey, Earl B. . . . . *C.E.*, '94 . . . . E. Aurora, N. Y.
Trav. Salesman, care John R. Keim, Buffalo, N. Y.
Bailey, John D. . . . . *C.E.*, '00 . . . . Dansville, N. Y.
Baker, Chas. H. . . . . *C.E.*, '86 . . . . Seattle, Wash.
President Snoqualmie Falls Power Co.
Baker, Howard W. . . . . *C.E.*, 86 . . . . 230–236 Adams St., Chicago, Ill.
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Balcom, H. Gage . . . . *C.E.*, '97 . . . . . . Bergen, Genesee Co., N. Y.
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Barbour, Irwin W. . . . . . . *C.E.*, '94 . . . . Woodfords, Me.
Asst. Engr. Dept. Public Wks., Portland, Me.
Bardol, Frank V. E. . . . . . . *C.E.*, '89 ; M. Eng. Soc. W. N. Y. ; 437 E. Utica St.
Ch. Engr., Board Public Works, 13 City Hall, Buffalo, N. Y.
Barnes, Fred A. . . . . *C.E.*, '97, *M.C.E.*, '98 . . . . Stockbridge, Mass.
Asst. Engr., Dept. Santiago and Puerto Principe, Santiago de Cuba.
Barros, Carlos Paes de . . . . *C.E.*, '76 ; 18 Rua Santa Ephigenica,
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Barton, Calvin L. . . . . *C.E.*, '99 . . . . Little Meadows, Pa.
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Bassett, Herbert H., ... *C.E.*, '00 . . . 42 Bassett St., New Britain, Conn.
Battin, Henry W. . . . . *C.E.*, '81 . . . . Baraboo, Wis.
Asst. Supt., C. & N. W. Ry.
von Bayer, August H. . . . . *C.E.*, '00 . 2016 13th St., N. W., Washington, D. C.
Beahan, Willard . . . . *C.E.*, '78 ; M. Am. Soc. C. E., M. St. L. E. Club.
Div. Engr., L. V. R. R., Easton, Pa.
Bean, Milford C. . . . . *C.E.*, '72 . . . . McGrawville, N. Y.
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Beardsley, James W. . . . . . *C.E.*, '91 ; M. West. Soc. C. E.
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Becker, Charton L. . . . . . . *C.E.*, '88 . . . . Sterlingville, N. Y.
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Belden, Clifford H. . . . . . . *C.E.*, '99 . . . . 217 Laurel St., Hartford, Conn.
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Benson, Orville . . . . . . *C.E.*, '88 ; M. O. Soc. Surv. and C. E. ; Canton, O.
Engr. Ch. Pittsburg Works Berlin Bridge Co., Economy, Pa.
Bertolet, Heyman E. . . . . . *C.E.*, '99 . . . . Oley, Pa.
Asst. Isthmian Canal Com., Greytown.
Beye, John C. . . . . *C.E.*, '83 ; M. West. Soc. Engrs. ; Kansas City, Mo.
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Clay, Francis W. H. . . . . . *C.E.*, '93, *LL.B.*, *LL.M.*, Cincinnati, O.
Lawyer, Specialty Patents, Carlisle Bldg.
Coe, Ira J. . . . . *C.E.*, '94 . . . . Dover, N. J.
Civil Engineer, 210 Water St., New York.
Coit, Charles W. . . . . *C.E.*, '00 . . . . . . 7 Clark St., Holyoke, Mass.
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Supt. Bridges and Bldgs., S. P. R.R. Co., Atlantic System, Houston.
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Contractor, 110 Laclede Bldg., St. Louis, Mo.
Colsten, Albert L. . . . . *C.E.*, '95 . . . . Port Deposit, Md.
Master in Applied Mathematics, Tome Institute.
Comstock, Charles W. . . . . *C.E.*, *Met.E.*, *M.C.E.*, '94, *Ph.D.*, '98; Assoc. M. Am. Soc. C. E., Secy. Col. Scientific Soc.; 278 S. Lincoln Ave., Denver, Colo.
Prof. Mining Eng., Colorado State School of Mines, Golden.
Conable, Morris R. . . . . *C.E.*, '76, *M.S.* . . . 82 E. 5th St., St. Paul, Minn.
Conger, Alger A. . . . . *C.E.*, '97 ; Jun. Wes. Soc. Engrs.; Gouveneur, N. Y.
Instrument man, Isthmian Canal Commission.
Conklin, William E. . . . . . . *C.E.*, '00 . . . . Fishkill, N. Y.
Conkling, Leon D. . . . . *C.E.*, '00 . . 613 W. Water St., Elmira, N. Y.
Asst. City Eng., Elmira, N. Y.
Cornell, Oliver H. P. . . . *M.C.E.*, '74 . . . . Ithaca, N. Y.
President Standard Barrel Co., 23 Park Row, New York.
Cory, Harry T., . . *B.M.E.*, *B.C.E.*, *M.C.E.*, '93 ; Assoc. M. Am. Soc. C. E., Jun. Am. Soc. M. E., Sec. Asso. Univ. of Mo.
Prof. Civil Engrg., Univ. of Mo., Columbia, Mo.
Couch, Vinton M. . . . . *C.E.*, '92 . . . . Odessa, N. Y.
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Civil Engr., Scranton Gas & Water Co., Scranton, Pa.
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Prof. Railway Engineering, Cornell University, Ithaca, N. Y.
Crane, Albert S. . . . . *C.E.*, 91, ; Assoc. M. Am. Soc. C. E., M. Bos. Soc. C. E., M. Brook. E. Club ; Addison, N. Y.
Ch. Asst. Engr., Mich. Lake Superior Power Co., Sault Ste. Marie, Mich.
Crouch, Nelson S. . . . . . *C.E.*, '90 . . . . Erie, Pa.
Miller and Grain Dealer, 240 E. 8th St.
Cummings, Elmore D. . . . . *C.E.*, '89 . . . . Indiana, Pa.
Draftsman, U. S. Engrs. Office, St. Paul, Minn.
Cummings, Noah . . . . *C.E.*, '94 ; Jun. Am. Soc. C. E. . Chaseville, N. Y.
Asst. Engr., Board Pub. Impvts., 3d Ave. and 177th St., New York.
Cunningham, William J. . . . *C.E.*, '00 . . . . . Sag Harbor, Long Island, N. Y.

Curtis, Charles E. . . . . . *C.E.*, '85 . . . . . . 217 Cedar St., Corning, N. Y.
With Cambria Steel Co., Johnstown, Pa.
Curtis, Charles W., ... *C.E.*, '88, *LL.B.* . . . . Washington, D. C.
Advertising Manager, Sill Stove Works, Rochester, N. Y.
Curtis, Gram . . . . . *C.E.*, '72 . . . . 60 Wallace Ave., New Castle, Pa.
Engineer, National Steel Co., New Castle Works.
Curtis, Winthrop L. . . . . . *C.E.*, '92 . . . . Horseheads, N. Y.
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Asst. Engr., San. Dist., Chicago, Office 41 at 5502 Halsted St., Chicago, Ill.
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Chief Engr., Massillon Bridge Co., 520 Gardiner Bldg.
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Davis, Lynn L. . . . . *C.E.*, '96 . . . . Clear Creek, N. Y.
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Taylor, T. Walter . . . . . *C.E.*, '00 . . . . 62 Rush St., Brooklyn, N. Y.

Tenney, Maynard A. . . . . . *C.E.*, '98 . . . . . 17 Hamilton Ave., Lynn, Mass.
Traveling in Europe.

Terrell, Adelphus C. . . . . . *C.E.*, '00 . . . . Macon, Mo.

Thacher, Cornelius S. . . . . *C.E.*, '78 . . . . . 364 Clifton Ave., Newark, N. J.
Prof. Math., Newark High School.

Thebaud, John E. . . . . . . *C.E.*, '95 . . . . 763 7th St., Buffalo, N. Y.
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Thomas, Howard . . . . *C.E.*, '77 . . . . West Superior, Wis.
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Thomas, Seymour P. . . . . . *C.E.*, '72 . . . . 49 William St., New York.
Resident Engineer, Phœnix Bridge Co.; N. Y. Mgr., Phœnix Iron Co.

Thompson, Ellis D. . . . . . *C.E.*, '76; M. Am. Soc. C. E., M. E. Club, Phila.
U. S. Asst. Engr., 815 Witherspoon Bldg., Philadelphia, Pa.

Thomson, Alexander, Jr. . . . *C.E.*, '99 . . 517 Lexington Ave., Brooklyn, N. Y.
Draftsman, Cent. R.R. of N. J., 143 Liberty St., New York.

Throop, William B. . . . . . *C.E.*, '77; M. W. Soc. Engrs., M. W. Ry. Club.
Div. Supt., C. B. & Q. R.R., Galesburg, Ill.

Tibbets, Addison S. . . . . . *C.E.*, '77 . . . . . 658 So. 17th St., Lincoln, Neb.
Attorney at Law.

Tier, Lewis P. . . . . *C.E.*, '74 . . . . 54 S. Linwood Ave., Norwalk, O.
Engine Dispatcher, Tol. Div. L. S. & M. S. Ry.

Tilton, Benjamin E. . . . . . *C.E.*, '97 . . . . Oshkosk, Wis.
Aid, U. S. C. and G. Survey, Washington, D. C.

Tomlinson, Frank C. . . . . *C.E.*, '74 . . . . . 222 Park Ave., Ironton, Ohio.
Cashier Second National Bank.

Tompkins, George S. . . . . *C.E.*, '96 . . . . 190 E. 17th St., Brooklyn, N. Y.
Inspr. for Mid. States Insp. Bur., 58 William St., New York.

Torrance, Chester C. . . . . *C.E.*, '99; *M.C.E.*, '00 . . . . . . Gowanda, N. Y.
Graduate Scholar, Civil Engineering, Cornell University.

Torrance, William M. . . . . *C.E.*, '95 . . . . Gowanda, N. Y.
Asst. Engr., Easton & Amboy R.R., Lehigh Div., L. V. R. R., Easton Pa.

Towl, Forrest M. . . . . . . *C.E.*, '86; M. Am. Soc. C. E., M. A. S. M. E.
Engr. Nat. Transit Co., 26 Broadway, New York.

Towle, John W. . . . . *C.E.*, '94 . . . . Falls City, Neb.
Gen. Wes. Agt., Canton Bridge Co., 300 Bee Bldg., Omaha, Neb.

Trautwine, John C., 3rd . . . . *C.E.*, '00 . . . 1111 Walnut St., Philadelphia, Pa.

Truesdell, Walter E. . . . . . *C.E.*, '97; M. Conn. As. C. E. & Sur.;
Packerville, Conn.
Civil Engr., 608–611 Mutual Reserve Bldg., New York.

Trumbull, William C. . . . *C.E.*, '82 . . . Gold Basin, Snohomish Co., Wash.

Truran, Ernest A. . . . . . . *C.E.*, '95 . . . 156 Washington St., Elmira, N. Y.
Draftsman, Elmira Br. Co., Ltd.

Turneaure, Frederick E. . . . *C.E.*, '89; Assoc. Am. Soc. C. E., M. W. Soc. E.
Prof. Bridge and San. Engrg., Univ. of Wis., Madison, Wis.

Turner, Ebenezer T. . . . . . *C.E.*, '83 . . . . Ithaca, N. Y.
Observer, U. S. Weather Bureau.

Turner, Horace G. . . . . . *C.E.*, '92 . . . . Pope's Mills, N. Y.
Engr. of Const., Town & Wallace, 1016 Townsend Bldg., New York.
Twining, William . . . . . . *C.E.*, '90 . . . . E. Mauch Chunk, Pa.
Asst. Engr., C. R. R. of N. J., Jersey City, N. J.
Underhill, Arthur . . . . . *C.E.*, '99 . . . 107 Harrison Ave., Montclair, N. J.
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Upjohn, Richard R. . . . . . *C.E.*, '80, *B.D.* . 296 Clinton St., Brooklyn, N. Y.
Priest.
Vedder, Herman K. . . . . . *C.E.*, '87.
Professor Math. and Civil Eng., Agricultural College, Mich.
Vedder, Wellington R. . . . . *C.E.*, '91 . . . . Leeds, N. Y.
Asst. City Engr., City Hall, Syracuse, N. Y.
Vickers, Thomas McE. . . . . *C.E.*, '90 ; *M.C.E.*, '91 ; Assoc. M. Am. Soc. C. E.
Engr., Dept. of Pub. Wks., Bur. of Water, 137 N. State St., Syracuse, N. Y.
Vogleson, John A. . . . . . . *C.E.*, '00 . . . . Columbiana, O.
Vose, Walter I. . . . . *C.E.*, '92 . . . . Manville, R. I.
Merchant.
Wadsworth, Joel E. . . . . . *C.E.*, '90 ; Assoc. M. Am. Soc. C.E., M. S. P. E. Ed.
Asst. Engr., Berlin Iron Br. Co., Middletown, Conn.
Waesche, George E. . . *A.B.*, *C.E.*, '95 . . . . Thurmont, Md.
Instructor, Civil Engrg., Purdue Univ., Lafayette, Ind.
Wagner, George O. . . . . . *C.E.*, '00 . . . . 160 N. Pearl St., Buffalo, N. Y.
New Steel Works, Buffalo.
Wait, John C. . . . . *C.E.*, '82, *M.C.E.*, '91, *LL.B.*, ; M. Am. Soc. C. E.
Counsr. at Law (Eng. and Arch. Jurisprudence), 220 Broadway, New York.
Wait, Owen A. . . . . *C.E.*, '98.
Draftsman, Niagara Falls Power Co., 631 Erie Ave., Niagara Falls, N. Y.
Wallhauser. George O. . . . . *C.E.*, '96 . . . . 602 E. State St., Olean, N. Y.
Arkansas Northern Ry., Springfield, Mo.
Warner, Monroe . . . . *C.E.*, '88 . . . . Pulaski, N. Y.
Supt. Constr., Elect. Rys., Sanderson & Porter Co., Putnam, Conn.
Warriner, Thomas B. . . . . *C.E.*, '93 ; M. Ia. Eng. Soc.
Civil Engineer, Cedar Rapids, Ia.
Warthorst, Frank W. . . . . *C.E.*, '74 . . . . *Rosedale, Kern Co., Cal.*
Warthorst & Co., Mfrs. and Quarrymen, Masillon, O.
Washburn, Frank S. . . . . . *C.E.*, '83 ; M. Am. Soc. C.E., M. West. Soc. C.E.
Nashville, Tenn.
Wasson, Charles W. . . . . . *C.E.*, '74 ; M. Acad. Sci.
Principal Graded School, Elmira, N. Y.
Weatherson, John . . . . . . *C.E.*, '95, *M.D.* . . 3739 Prairie Ave., Chicago, Ill.
Physician, cor. 43rd St. and Lake Ave.
Webb, Walter L. . . . . . . *C.E.*, '84, *M.C.E.*, '89 ; Assoc. M. Am. Soc. C. ,E. M. E. Cl., Phila.
Asst. Prof. Civil Engrg., College Hall, Univ. of Pa., Philadelphia, Pa.
Weed, Addison . . . . *C.E.*, '79 . . . . New Hartford, N. Y.
Farmer, North Rose, N. Y.
Welker, Philip A. . . . . . . *C.E.*, '78 ; M. Nat. Geog. Soc. . Washington, D. C.
Asst. U. S. Coast and Geod. Survey, Commanding U. S. S. "Bache."

Wheelock, Charles B. . . . . *C.E.*, '76 . . . . Boston, Mass.
Insurance, 93 Water St.
Whelpley, J. R. . . . . *C.E.*, '96 . . 800 East Cap. St., Washington, D. C.
Rodman, Engr. Dept., D. C.
Whiskeman, James P. . . . . *C.E.*, '98 . . . . . . 30 So. 4th St., Reading, Pa.
Asst. Engr., Ch. Engr's. Of., L. & N. R.R., Louisville, Ky.
White, Timothy S. . . . . . *C.E.*, '73 ; M. Am. Soc. C. E., M. E. Soc. W. Pa.
Vice-President and Ch. Engr., Penn. Br. Co., Beaver Falls, Pa.
White, Willard O. . . . . *C.E.*, '00 . . . . Blossvale, N. Y.
Whitson, Abraham U. . . . . *C.E.*, '99 . . . . Westbury Sta., L. I., N. Y.
Topo. Drft., Board Pub. Impvts. of N. Y., 21 Jackson Ave., Long Island City.
Wick, Richard B. . . . . *C.E.*, '81 . . . . Warren, Ohio.
Engr. and Treas., The Miller Const. Co., Belington, W. Va.
Wilcox, Robert B. . . . . . . *C.E.*, '90 . . . . 285 W. Adams St., Chicago, Ill.
Engr. in Charge of Harbor, City of Chicago, 325 City Hall.
Williams, Chauncey G. . . . *C.E.*, '87 ; Assoc. M. Am. Soc. C. E.
117 Montague St., Brooklyn, N. Y.
Williams, Friend P. . . . . . *C.E.*, '99 . . . . 703 W. Sullivan St., Olean, N. Y.
Engrg. Corps, Pittsburg, Shawmut & North. R.R., Redlands, Cal.
Williams, Sylvester N. . . . *C.E.*, '72 ; M. Soc. Pro. Engrg. Ed., M. I. E. Soc.
Prof. of Civil Engineering, Cornell College, Mt. Vernon, Ia.
Windsor, Philip B. . . . . . . *C.E.*, '00 . . . . . . "Echobank," Swains, N. Y.
Wing, Charles B. . . . . *C.E.*, '86 ; Assoc. M. Am. Soc. C.E., M. T. Soc. P. Cst.
Prof. Structural Engrg., Stanford Univ., Palo Alto, Cal.
Wing, Frederick K. . . . . . *C.E.*, '90 ; Assoc. M. Am. Soc. C. E.
Civil Engineer, 111 White Building, Buffalo, N. Y.
Wolfe, Frank C. . . . . *C.E.*, '95 . . . . Union Bridge, Md.
With American Bridge Works, 40th St. and Stewart Ave., Chicago, Ill.
Works, Norris M. . . . . *C.E.*, '97 . . . . Lima, N. Y.
U. S. Sub. Inspr., Buffalo Breakwater Const., 1100 Morgan Building.
Young, Henry A. . . . . *C.E.*, '99 . . . 110 Ravine Ave., Yonkers, N. Y.
Transitman, Tacon 3, Dept. of Havana, Havana, Cuba.
Zarbell, Elmer . . . . *C.E.*, 95 ; Assoc. M. Am. Soc. C. E.; 4132 Ellis Ave.
Asst. Engr., Special Comrs., Chic. Drainage Canal, Chicago, Ill.

DECEASED MEMBERS.

NAME.		RESIDENCE.	DATE OF DEATH.
Ames, Willis C.	*C.E.*, '77 ;	Whitney's Point, N. Y.,	Feb. 23, 1894
Aylen, Charles P.	*C.E.*, '76 ;	Aylmer, Canada,	April, 1894
Bueno, Francisco de A. V.	*C.E.*, '76 ;	Rio de Janerio, Brazil,	About 1881
Carpenter, Frank DeY.	*C.E.*, '73 ;	*M.C.E.*, '76 ; Highland, N. Y.,	Dec. 19, 1883
Clark, Frank B.	*C.E.*, '96 ;	Fulton, N. Y.,	Oct. 29, 1899
Clark, Ira E.	*C.E.*, '72 ;	Weston, Mass.,	May 23, 1882
Cook, Isaac N.	*C.E.*, '75 ;	Jersey City, N. J.,	May 7, 1885
Cooper, Edgar H.	*C.E.*, '85 ;	New York City,	Oct., 1890
Dobraluboff, John A.	*C.E.*, '74 ;	Nijney Novgorod, Russia,	About 1882
Dodd, Franklin M. G.	*C.E.*, '90 ;	Franklin, N. J.,	Sept. 13, 1891
Doerflinger, Augustus	*C.E.*, '71 ;	Brooklyn, N. Y.,	Nov. 24, 1899
Eidlitz, Alfred F.	*C.E.*, '76 ;	New York City,	April 22, 1877
Farnham, Whitfield	*C.E.*, '71 ;	*M.C.E.*, '74 ; St. Louis, Mo.,	April 13, 1895
Ferguson, Nicholas C.	*C.E.*, '79 ;	St. Louis, Mo.,	Sept. 22, 1896
Fitch, William R.	*C.E.*, '74 ;	Ithaca, N. Y.,	April 14, 1886
Foster, Reuben B.	*C.E.*, '74 ;	*M.C.E.*, '77 ; S. Lake Weir, Fla.,	Nov. 7, 1895
Frost, Frederic W.	*C.E.*, '72 ;	New York City,	Oct. 3, 1899
Greene, Almon C.	*C.E.*, '75 ;	Palmyra, N. Y.,	July 28, 1897
Gunner, Daniel W.	*C.E.*, '87 ;	Schagticoke, N. Y.,	Oct. 10, 1887
Gibbs, Harley S.	*C.E.*, '98 ;	Pittsburg, Pa.,	Aug. 8, 1899
Hallock, E. Allen	*C.E.*, '91 ;	Moriches, N. Y.,	April 13, 1900
Holbrook, Ernest M.	*C.E.*, '89 ;	*M.C.E.*, '90 ; Ithaca, N. Y.,	Oct. 9, 1892
Hulse, Howard C.	*C.E.*, '91 ;	Brooklyn, N. Y.,	Feb. 20, 1893
Landers, Herbert H.	*C.E.*, '90 ;	Green Island, N. Y.,	Feb. 4, 1893
Lyman, George F.	*C.E.*, '73 ;	Tenafly, N. J.,	Dec. 25, 1880
MacMullen, Justus C.	*C.E.*, '76 ;	Unionville, N. Y.,	Jan. 31, 1888
Preston, Kolce	*C.E.*, '73 ;	Wilmington, Del.,	Jan. 4, 1876
Rogers, Jesse A.	*C.E.*, '91 ;	Evans Mills, N. Y.,	April 24, 1897
Sheldon, Daniel C.	*C.E.*, '83 ;	Delphi, N. Y.,	Oct. 2, 1893
Shepard, Frank W.	*C.E.*, '86 ;	Medina, O.,	Feb. 10, 1892
Smith, George La T.	*C.E.*, '71 ;	*M.C.E.*, '74 ; Canandaigua, N.Y.,	June 25, 1892
Smith, William J.	*C.E.*, '79 ;	Charleston, N. Y.,	Dec. 3, 1886
Stewart, Neil, Jr.	*C.E.*, '87 ;	York, N. Y.,	March 30, 1891
Tilley, George A.	*C.E.*, '73 ;	Washington, D. C.,	March 14, 1877
Tompkins, John H.	*C.E.*, '73 ;	Poughkeepsie, N. Y.,	July, 1879
Van Cauteren, Emile A.	*C.E.*, 97 ;	Pringy, Seine-et-Marne, France,	July 4, 1899
Viegas-Muniz, Joaquim	*C.E.*, '77 ;	Piracicaba, Brazil,	About 1883
Wightman, Willard H.	*C.E.*, '81 ;	Ashland, Ore.,	Oct. 29, 1889

IN MEMORIAM.

AUGUSTUS DOERFLINGER, C.E., '71.

Augustus Doerflinger, C.E., was a descendant of an old German family, for several generations resident in Lorraine, France. He was born in Baden, Germany, in 1845, and came to this country with his parents when three years of age.

He was graduated from Cornell University in '71, and in the same year entered the Engineer Corps of U. S. Army as Assistant Engineer. The greater part of his professional career was spent in this service. He was stationed at the New York office and from '71 to '77 was engaged principally upon the improvements of the East River. This included the celebrated destruction of the reefs at Hell Gate. Together with the late J. H. Streitinger, also a U. S. Assistant Engineer, he devised the arrangements which resulted in the successful disruption of the reef at Hallet's Point on September 24, 1876. For this work they received a gold medal of honor from the government of France and a decoration from the Belgian government. Subsequently he was occupied in the engineering works on the approaches to New York Harbor. Among these were the widening and deepening of the Gedney and main ship channels, and the preservation of the beach at Sandy Hook from the encroachment of the sea. Later he was the engineer officer locally in charge of the batteries at Fort Hamilton, and was then assigned to the improvements of the Harlem River. He was continuously engaged upon this work, culminating in the opening of the ship canal in 1895.

After the cessation of the work on the Harlem , Mr. Doerflinger engaged in the general supervision and study of the many problems which presented themselves at the Engineer office in New York. He was connected as consulting engineer with the construction of many of the modern buildings of the city and had the privilege of contributing his services as the engineer for the building of the Grant Memorial on Riverside Heights. Later the first projects for the great 35 foot ship channel from the Narrows to the sea were in his charge. Participation in the active work of this great undertaking was prevented by his illness which resulted in his death, November 24, 1899, at the age of fifty-four.

Mr. Doerflinger was a man of retiring habits and extreme modesty. His interests were all centered in his home and his work. He was highly esteemed by all with whom he came in contact, and all who knew him will voice the words of General Ludlow, Military Governor of Havana, who wrote—"Will you take occasion to present to Mr. Doerflinger's family and personal friends the very sincere regret I feel in reading a notice of his death and an expression of my very high appreciation of his personal character and professional attainments and services. I knew him long and well and no one stood higher in my esteem and regard."

FREDERICK WILLIAM FROST, B.C.E. '72.

Our alumni has suffered a heavy loss by the death of Frederick W. Frost at Seattle, Wash., October 3, 1899. He was making his fifth trip to Alaska to examine some mining interests, when he was suddenly taken ill at Seattle. This illness proved fatal and the postmortem revealed an aneurism of the aortic artery.

Mr. Frost was born in Springfield, Mass., Jan. 8, 1852. He graduated from the High School when 14 years old and after being in business in his native city for two years, entered Cornell with the Class of '72. After graduation he served three years on canal location in Maryland, Pennsylvania and Virginia and three years in the surveying division of the U. S. General Land Office. Later he located the boundary line between Dakota and Wyoming, and as principal assistant engineer under the U. S. Engineer Corps he made surveys from Norfolk Harbor, Va., through the Dismal Swamp of Virginia to Cape Fear River, N. C. For seven years he was special assistant engineer on the construction of the Croton Aqueduct, in connection with this work he devised a good practical formula for computing the flow of water described in his valuable paper entitled "The Flow of Water in the New Croton Aqueduct." Other work on which he was engaged included a season's work in railroad construction on the Panhandle of Texas, a professional visit to the Panama Canal, Assistant Engineer in Department of Public Works, New York City, a year at the Brooklyn Dry Dock, sinking the iron columns for Atlantic City Promenade by hydraulic jet, and at the time of his death in promoting the development of Alaskan interests. Mr. Frost was honored by being selected from over a thousand applicants to receive the offer of the position of division engineer from the Nicaragua Canal Commission, his Alaskan trips prevented his accepting.

From the above it will be seen that Mr. Frost lived a very active and useful life. That his worth was recognized by the engineering profession is shown by the report of the Board of Expert Engineers on the New Croton Dam, specially commending "Mr. Frederick W. Frost, C.E., for the computations made, involving an immense amount of mathematical work which has been most faithfully and intelligently performed by him."

Mr. Frost was married in 1881 to Miss E. Olivia Ford of Norfolk, Va. His widow and two children, Frederick W., Jr. and Florence E., survive him, and to them we extend our sympathy.

ELIJAH ALLEN HALLOCK, C.E., '91.

E. A. Hallock was born at Moriches, Long Island, in 1871. After preparing at Prof. Hallock's Literary and Commercial Institute at Bridgehampton, L. I., he entered Cornell with the Class of '91, when but sixteen years of age. After graduation he accepted a position with the Pennsylvania Bridge Co. of Beaver Falls, Pa. After a few months he contracted typhoid fever, and had to leave. The following spring he secured a position with the Massillon Bridge Co., of Massillon, O. In the summer of 1894, he married Miss Bertha L. Merwin. In the fall of 1896, he held a position in Springfield, Mass., but consideration for his wife's health induced him to surrender it and go south to live. In the fall of 1898, both being in poor health, they went to Colorado. The continuation of poor health induced them to return home in March, 1899. He survived the trip but three weeks, and his wife died three weeks later. They leave a son, Merwin Fanning, three years of age.

Mr. Hallock's life was remarkable for the fortitude with which he passed through most bitter and trying experiences, and the persistence of his ambition to rise in his chosen profession. Despite all his sickness, he was very successful, and was frequently promoted. His physician writes, "I was much impressed with his great fortitude, and resignation to an inevitable end." Our deepest sympathy is extended to his widowed mother.

FRANK BERRY CLARK, '96.

The friends of Frank Berry Clark, were shocked to learn of his sad drowning while on the Isthmian Canal Survey on October 29, 1899.

Mr. Clark's preparatory education was received at Westport High School, where he graduated in 1891. That year, by competitive examination, he procured a State Scholarship and entered Cornell University with the Class of '95, but owing to illness was obliged to leave College in the early part of the Winter Term.

Mr. Clark had chosen for his profession Civil Engineering and so entered again in 1892 and took his degree with the Class of '96. Mr. Clark's work in College, like everything else he did, was thorough, conscientious, methodical.

Shortly after graduation, Mr. Clark married Miss Mamie Kimber of Fulton, New York, who was also graduated with the Class of '96.

For two summers before and one after graduation, Mr. Clark was surveying in the Adirondacks under Mr. Verplank Colvin, where he received steady promotion. He also spent two successful years on the Erie Canal, leaving with the termination of this work.

Last August he received a position on the Isthmian Canal Survey, only to lose it by death in two short months. But that in this brief period Mr. Clark's ability and worthy qualities had made themselves felt, is shown by the following letter :

"GREYTOWN HEADQUARTERS, Nov. 21st, 1899.

By order of the President of the Isthmian Canal Commission, dated the 27th of October, 1899, Mr. Frank B. Clark was promoted to the grade of Senior Assistant Engineer, the promotion to date from October 1st, 1899.

Although a deplorable accident caused the death of Mr. Clark on the 29th of October, the deceased officer's estate will of course receive the increased salary for October ; and this matter is published especially that it may be known that Mr. Clark's great worth was appreciated by the Isthmian Canal Commission and by those with whom he worked.

J. IMBRIE MILLER,
Chief Engineer of Surveys."

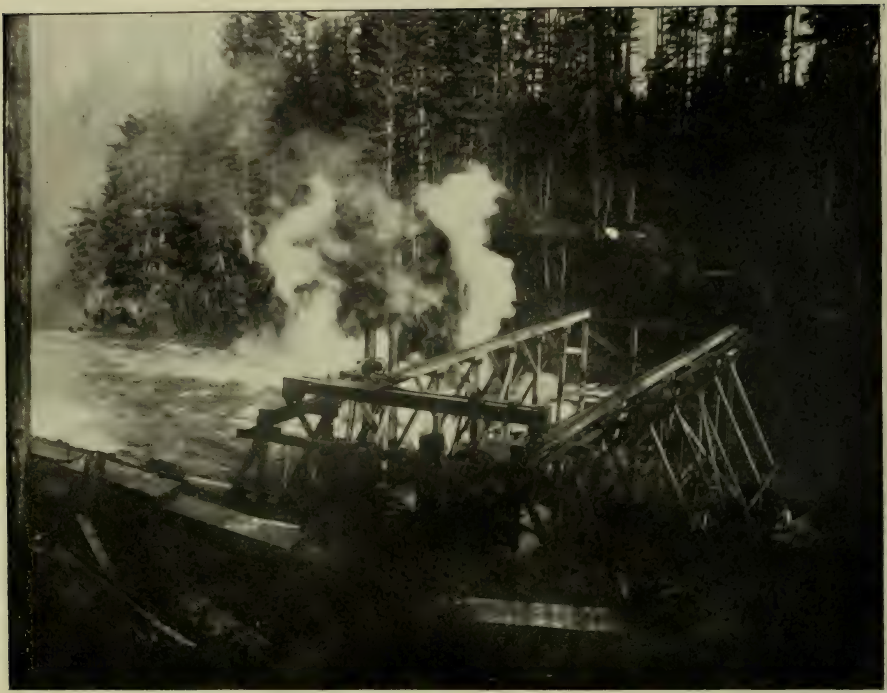
The death of Mr. Clark means the loss of a loyal Cornellian, a true friend and a noble man.

HARLEY STUART GIBBS, C.E., '98.

Harley Stuart Gibbs, died in Pittsburg, Pa., August 9, 1899. His fellow classmates and professors will remember him as a remarkable student, and a noble fellow. He took an active part in the Civil Engineering interests of the University, and was elected President of the Association of Civil Engineers.

He was born May 7, 1876, in Milan, Ohio. Three years later his family moved to Kansas, where he received a public school education. In 1892 the family moved to Elmira, N. Y. After spending two years at the Elmira Academy he obtained a Cornell University State Scholarship, passing the examinations with the highest average attained in the state. His ability was evident from the first, and as the reward of the high standard of his work maintained during the freshman and sophomore years, he received the Horace Greeley Scholarship. In the senior year he was admitted to Sigma Xi, and also received the Fuertes medal.

After graduating he accepted a position in Pittsburg, with John Ostrom, a bridge engineer and contractor. After a short year of service in this position, in which he demonstrated his ability as a practical engineer as well as a student, he contracted typhoid fever, and died after an illness of two weeks. Such is the brief history of a brilliant student, a true friend, and a noble man.



CONSTRUCTION PLANT, Etc.

SNOQUALMIE FALLS POWER TRANSMISSION.

CHARLES H. BAKER, C. U. '86. PRESIDENT OF SNOQUALMIE FALLS POWER COMPANY.

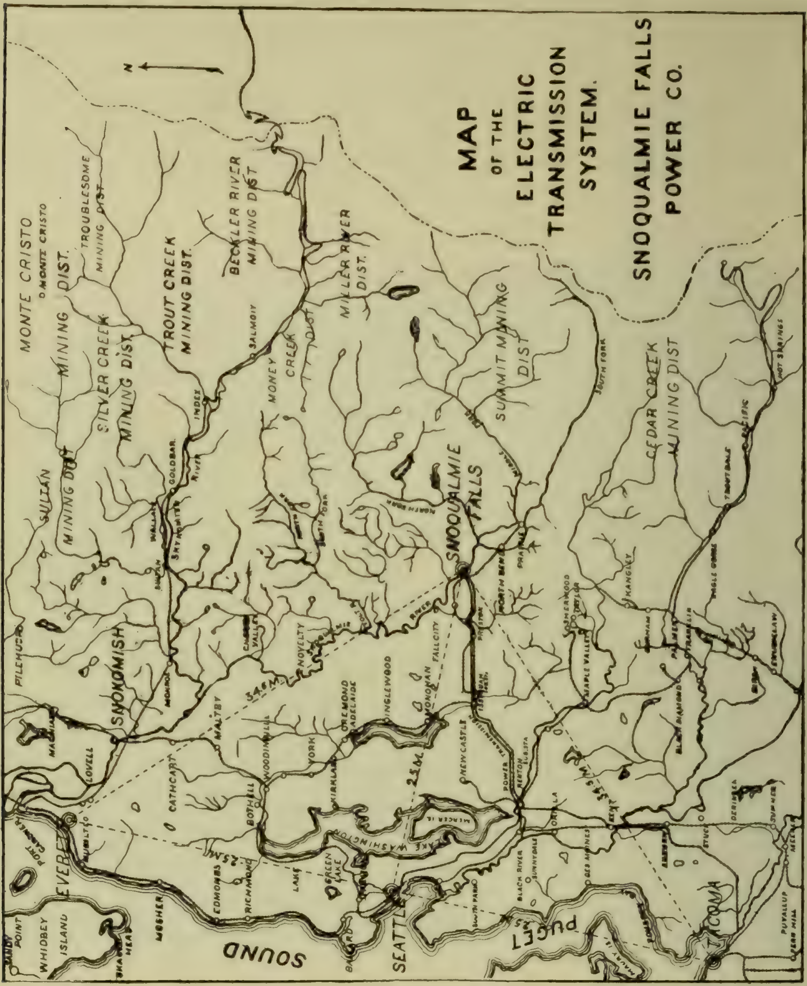
At the apex of an isosceles triangle having for its base a straight line drawn through the cities of Tacoma, Seattle and Everett, with Seattle midway, is situated the famous Snoqualmie Cataract. (See accompanying map.) About twenty-five miles back from the shores of Puget Sound in the foot-hills of the Cascade Mountains, this magnificent spectacle in nature has for years attracted the sightseer, but the utilitarian saw no beauty in the picture until the long distance transmission of power by electricity became possible. This waterfall and the adjacent land on both sides of Snoqualmie River were purchased by the writer in the fall of 1897. Early in 1898 the writer organized the Snoqualmie Falls Power Co., and soon after a heavy construction plant consisting of large boilers, steam hoisting machinery and a ten drill air compressor plant was installed at Snoqualmie Falls at the site of the proposed head works. The first drill began operating April 17th, 1898, and thereafter the work was prosecuted day and night and Sundays until the work was finally completed. It has been of incalculable advantage to the company having the Seattle & International Railway pass within fifty feet of the works. A heavy trestle was constructed spanning the railway, upon which a steel carrier is operated, by which means all the heavy machinery was lifted from the cars and transferred to the works.

The first water wheel and generator were in actual operation and delivered current into Seattle on the evening of July 31, 1899, and into Tacoma, November 1, 1899. This event marked the successful completion of a project which throughout had been unattended by any fatality or serious accident.

Original Features.—The design and execution of this plant is replete with original features, never before exploited in power transmission, conspicuous among them being:

1. Subterranean power house.
2. Snoqualmie Cycloidal Water Wheel, a very efficient water motor.
3. Use of aluminum wire in long distance transmission.

The successful completion and operation of the plant has demonstrated the entire merit of the boldest features.



WATERSHED.

The Snoqualmie watershed has an individuality peculiar to itself, which accounts for the large low water flow of the river as compared with other rivers on the western slope, the relative sizes of the drainage areas being taken into account.

Reconnaissance.—The Power Company, in the early stages of its operation, placed a party in the field for the purpose of exploring this watershed with a view of determining its hydrographic, geological, geographical, and meteorological features.

Geography.—The main Snoqualmie River is formed by three large forks originating at the summits of the Cascade Mountains and flowing westward to the point of confluence about three miles above Snoqualmie Falls. These rivers have their origin in the regions of perpetual snow and ice and are fed by tributary streams which also originate in lofty regions, so that the water is impounded upon the watershed in the shape of snow and ice, which serves to feed the river during the period of its lowest flow, in the hot and dry months of August and September. The entire watershed above the Falls is high in altitude. Beginning with the top of the Falls itself, which is about 600 feet above sea level, it slopes eastward and upward towards the mountain summits where the elevation averages a height of 8000 feet.

Rainfall.—The rainfall in this region is about three times that of the district immediately about the Sound, where, at Seattle for instance, the rainfall is 37 inches as against about 90 inches at the Falls. The meteorological reports show that the annual rainfall increases as the crest of the mountains is approached. The major part of the precipitation occurring in the Winter appears as snow, which melting away in the drier Summer months sustains a very considerable low water flow.

Disappearing Rivers.—The Company's explorers also noted the fact that the streams tributary to the forks of the main river disappear in the ground in some instances and come to the surface again in perhaps a half a mile or more. This physical characteristic of the watershed in having disappearing rivers sets forth the condition as that of a sponge, holding back in some measure the rainfall only to let it down during the dry season and period of low flow. The area of this watershed is not over 600 square miles and yet the river at the Falls runs from 1,000 cubic feet a second at its lowest stage to ten times that at its average stages. A river in the Mississippi Valley would require a watershed of ten times this area to produce an equal flow, which emphasizes the fact that the Snoqualmie River is remarkable for the amount of its discharge.

Impounding Reservoirs.—With so much water going to waste the idea is naturally suggested that means might be devised for equalizing the flow of the river and increasing the low water discharge. This suggestion finds its answer immediately in the researches of the exploring party who have found large lakes and natural reservoirs in the mountains, which can be dammed at small cost, so that the water impounded during the flood seasons can be let down during the dry months of August and September. The largest of the lakes are Lake Hancock and Lake Calligan. Although the water reaches as low a stage at the Falls as 30,000 H.P., the Company will be able to secure a uniform flow throughout the year of 100,000 H.P., by the utilization of the natural storage basins referred to. The power could be considerably augmented beyond this figure, if at some future time the industries of Seattle and Tacoma and other neighboring cities should require it, by the erection of a dam fifty feet high at the Falls. In this way a lake can be impounded behind the Falls about fifteen miles in area and of an average depth of twenty-five feet, which, in addition to the mountain storage, would increase the Company's ability to develop power throughout the year to perhaps 200,000 H.P. Probably at some remote geological time, a lake, such as has been suggested, was actually in existence behind Snoqualmie Falls, but it has been drained during the process of years by the wearing away of the rocky river bed and the lowering of the precipice over which the river leaps and forms the cataract. Snoqualmie River does not freeze so that floating ice and anchor ice are unknown.

Preliminary Investigations.—One of the first studies was to determine the geological formations at the Falls as a determining factor in the choice of plans of development, and to ascertain the character and volume of flow. The exposed rock surface in the vicinity, including the railroad cuts, faces of cliffs, and dry river banks, were carefully examined. It developed that a rock uplift, narrow in an east and west direction, extended north and south crossing the river at the site of the Falls and disappearing at the east within one fourth mile. The rock proved to be basaltic and of volcanic origin with no set cleavage, hard and non-absorbent, and apparently seamed off into great individual ledges. These conditions led to the original underground plan of development referred to later.

Measurement of Flow.—A topographical survey, accurate in detail was made covering a radius of a quarter of a mile from the Falls and including the river bottom. At the same time guages were set at convenient intervals above and below the falls, to determine the elevations of the river at the various points at all times, and to establish the slopes at the various levels. These guages have been read three times daily so that the company has an accurate history of the river for the past two years, from its own records in addition to less elaborate data for several years preceding from other sources. A clever cut and uniform cross-section of the river is obtained about one quarter mile above the falls, at which point the company established a station for measuring the velocity of the river at all stages of water. An exhaustive series of accurate measurements has been made here with a current meter.

Compiling the data thus acquired and letting $y =$ elevation of river, $x =$ discharge (cu. ft. per sec.) we find the law of the river to be a parabola with the following equation ; $x = \frac{y^2 - 106.8\,y + 2851.6}{.01}$.

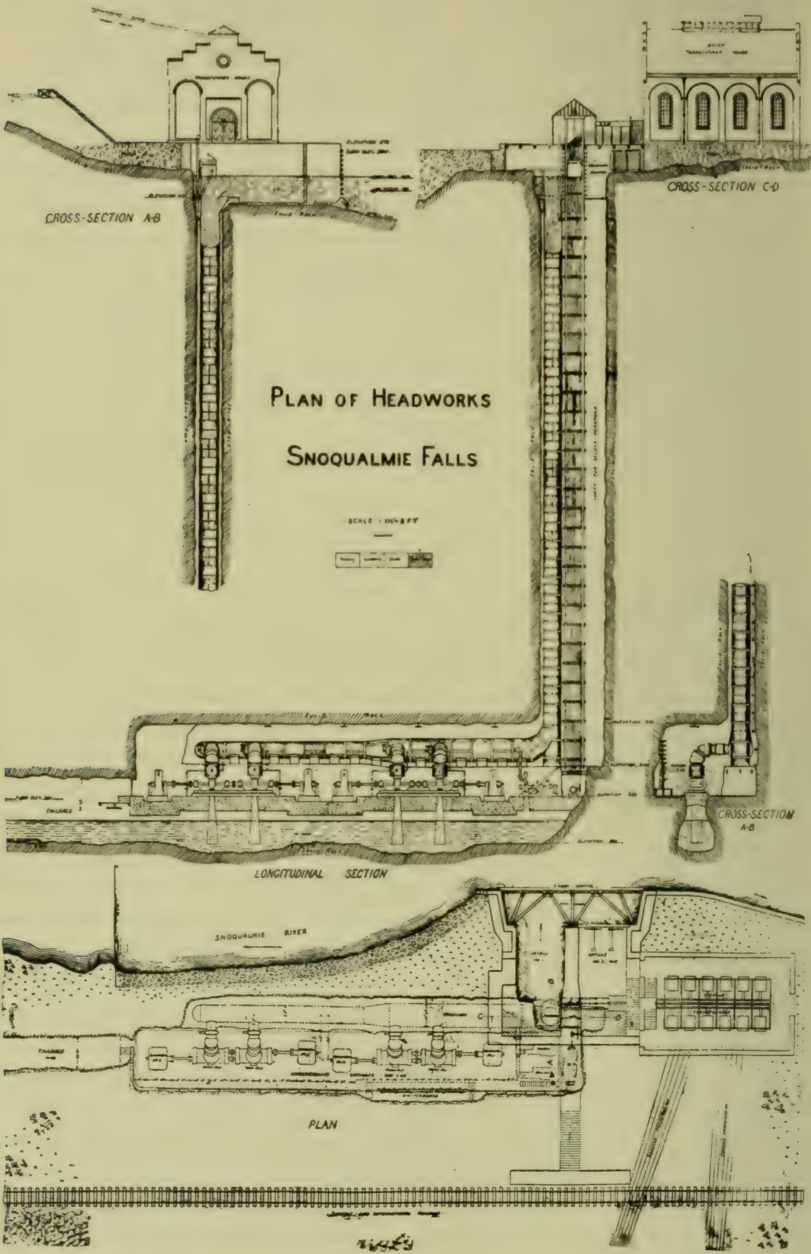
The river runs pure and crystalline at the Falls, except in a freshet when it carries some silt gathered from the valley immediately behind. White clay dissolved in the water is carried by the river at times. Freshets occur generally in November and June, the former due to excessive rainfall prevalent at that season, and the latter to the melting snows in the mountains. The river fluctuates between the high and low water stages throughout the year, with minor freshets interspersed, but the big flood precedes the dry season very closely, so that storage for a greater plant would not have to be long maintained.

Meteorological Station.—A daily record of rainfall, barometer, temperature and all conditions of weather, is kept by the company at Snoqualmie Falls, and is reported to the U. S. Weather Bureau at Seattle.

HEAD WORKS.

Shaft, Cavity and Tunnel—The hydraulic plan is simple and yet original. About 300 feet back from the brink of the falls, a shaft 10 ft. by 27 ft. in cross section, has been sunk in the bed of the river on the south side and descending 270 feet to the level of the river below the falls. Concurrent with the beginning of the shaft above the falls, a tunnel 12 ft. by 24 ft. with a slope of 2 feet in its length was drifted in from the face of the ledge below the falls, to an intersection with the shaft a distance of 650 feet. Beginning at the shaft and extending over and along the tunnel, a chamber 200 feet in length, 40 feet wide and 30 feet high, with the floor at the elevation of high water below the falls, was excavated out of the solid rock, and this cavity, nearly 300 feet below the surface of the earth, is the machinery room in which the water wheels and electric generators have since been installed. At average stages of the river the tunnel is submerged about two-thirds of its depth, while during flood seasons it is entirely submerged. The tunnel extends under the floor of the cavity, forming a tail race with a concrete roof. The water is carried down the shaft in a steel pipe which terminates in a horizontal steel pipe or receiver, and thence through water wheels into the tail race.

The walls of the chamber have been left rough and whitewashed, while the floor has been concreted. Five hundred incandescent lamps are used to light the shaft, chamber and tunnel.


CROSS-SECTION A-B
CROSS-SECTION C-D
PLAN OF HEADWORKS
SNOQUALMIE FALLS
SCALE
CROSS-SECTION A-B
LONGITUDINAL SECTION
SNOQUALMIE RIVER
TAILRACE
C
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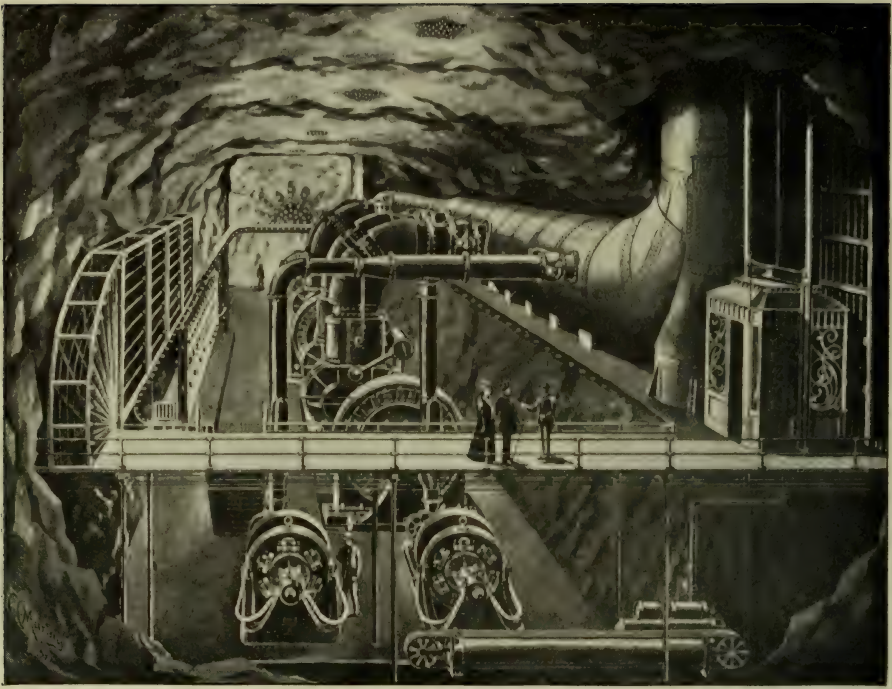
PLAN



CAVITY COMPLETE WITH MACHINERY INSTALLED.

This cavity is ventilated by natural draft through the tail race and up the shaft, which is so strong that it has to be curbed. The cavity is cool and perfectly dry, the temperature remaining the same throughout the year. This low and constant temperature contributes to higher efficiency in the generators and consequently a greater H.P. output. Here is a power house that needs no fire insurance, and where there is no depreciation ; while it is free from the frozen spray that clings in great icicles to the trees and cliffs in winter near the falls.

Intake.—The water is delivered from the river through an intake constructed of concrete masonry, the walls of which are six feet in thickness and 25 feet high, resting on the native rock. This bay is protected from the river by a steel girder construction, bearing against the outside and center masonry piers, and supporting a heavy grating of timbers from pier to pier across the opening of the intake. These timbers being 12 × 12 stringers, are laid horizontally with twelve inch spaces between them, through which the water flows into the intake. This timber grating protects the works from floating trees or logs, while just inside the intake are heavy steel wire screens of two inch mesh hung from the diagonal steel girders, which serve to stop any smaller debris from entering. All concrete was made of the rock excavation run through a stone crusher, and of river gravel, mixed with Portland cement and laid in six inch layers.

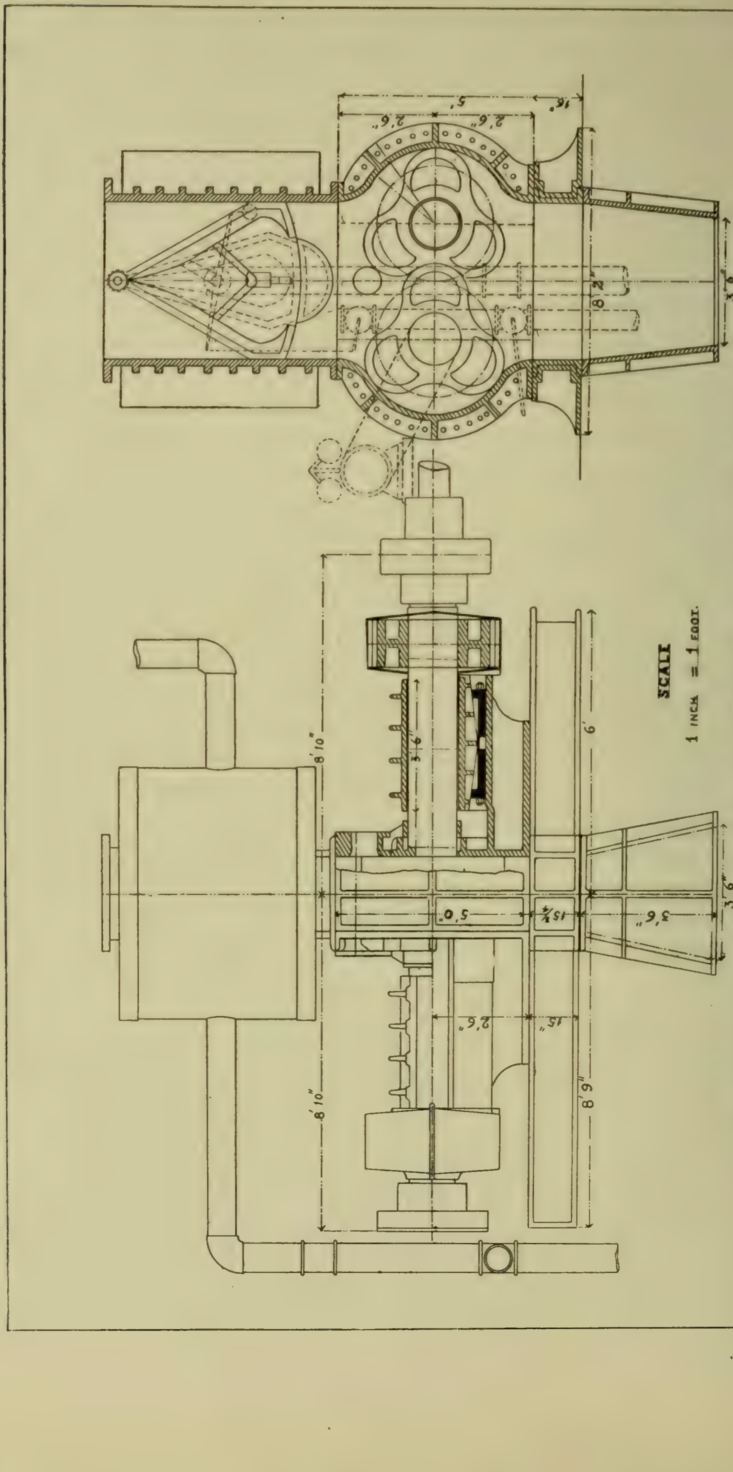
Protective Boom.—A rudder boom 300 feet in length is moored up stream on the intake side of the river and reaches past the intake. By turning the capstan at the head of the boom, the rudders are thrown out which causes the boom to swing out into midstream and serve as a fender for floating logs, etc. The river is 150 feet wide from the headbay to the opposite shore and about fifteen feet deep at ordinary stages and flows ten feet a second. The floor of the intake is on a submerged reef and is about five feet higher than the river bed.

Dam.—A concrete dam placed diagonally across the river from the down stream pier of the headbay to the opposite shore will be built at the next low water season and will have a length of about 400 feet in order to give a long crest and so avoid the backing up of flood waters. This dam which will be entirely submerged will raise the river level at the intake six feet and will permit of a better inflow, besides equalizing the daily flow of the river during the period of extreme low water.

Penstock.—A steel penstock 7½ feet in diameter rises through the excavated shaft to the bottom of the intake, the top or expansion joint of which is made tight with the rock walls of the shaft with concrete grout, so that a water tight roof is afforded to the shaft leaving the only escape for the water down through the penstock. A steel bulkhead 8 x 10 feet rises beside the penstock, and behind the center pier, through the intake and water to the open air above and connecting with the shaft space below, and this bulkhead, surmounted by a neat little house, is the front door of the subterranean power house. Through it the elevator travels for convenience in the operation of the works and through it the electric conductors ascend. The penstock descends 250 feet until the cavity is reached where it makes a right angle turn to a horizontal position, and enlarges to a diameter of 10 feet, reducing again to 8 feet at its end section, which horizontal construction is known as the receiver.

Receiver.—The receiver and lower half of the penstock are made of rolled steel, eight foot plates 1 inch thick. The upper half of the penstock is made of plates ⅞ inch thick at bottom and ½ inch at top. The whole construction is caulked bottle tight. The receiver lies upon a rock bench left in the cavity along its north margin rising 12 feet above the cavity floor. The penstock and receiver weigh 450,000 pounds, and the water column in the penstock weighs 340 tons. The shaft was excavated to carry two penstocks (in view of doubling capacity) leaving room for the elevator to travel between them, as well as space for the outgoing electric conductors. The elevator is operated hydraulically, through this central space the heavy water wheels and generators were lowered to the power house below. The receiver branches at four points along its length with four foot openings; each branch being opened or closed by a Rensaelaer valve weighing 23,000 lbs. Incidentally it may be remarked that these are the largest valves in the world operated under so great a pressure. Each branch is connected by a large cast iron elbow bending downward and connected with a 3,000 H.P. Snoqualmie Water motor resting upon the rock floor of the cavity below, under which the tailrace extends the entire length, to receive the discharge from the motors. Each water motor is directly connected with a 1,500 K.W. Westinghouse tri-phase generator. The current passes from the generators to the marble switchboard, and ascends the shaft on cables of twisted aluminum wires to the transformer house above at a voltage of 1,000. There is also a 24 inch branch from the receiver giving outlet for water to operate exciter motors; and also sundry small outlets for miscellaneous purposes.

Snoqualmie Cycloidal Water Motors.—The Snoqualmie water motor (see figure) is the invention of Mr. T. T. Johnston, C.E., of Chicago, and embodies the principle of the rotary pump. The moving parts of the motor are supported upon a heavy base frame (*H*) 8 x 15 feet, raising them five feet from the floor. Two shafts (*K*) lying parallel and each having a three lobed cycloidal impeller (*N*) at its center, revolve in the journal bearings (*E*) supported by the base in such a position that the impellers interlock. The revolving impellers are enclosed in a cast iron case (*G*) close fitting to the moving parts, the under part of which is open to the tail race to discharge the water, and the upper part is open to receive the water. The interlocking lobes fit to themselves and to the enclosing case, so nicely, that no water can pass through the motor without setting the impellers in motion, and following them in the recurring spaces. The two shafts supporting these impellers are geared together outside of the case at both ends, and the elongation of one of these shafts delivers the power to the generator to which it is connected. Each impeller shaft is 15 feet long, 16 inches in diameter and weighs 15,000 lbs., while the motor assembled weighs about 50 tons. Nothing simpler can be conceived, and due to its low cost and high efficiency, engineers predict for this water engine, now working in the bowels of the earth at Snoqualmie Falls, a bright and successful future. The motor revolves with a speed of 300 revolutions per minute and has a capacity of 3000 H. P. in the water column. The efficiency of a much smaller motor of the same design was found by test to be 90 per cent. It is estimated that for a motor of this size the efficiency would fall to about 80 per cent. It is simple in construction, cheaply built and can be designed for any head. Draft tubes connected with the motors, descend into the tailrace, thus securing advantage of the additional suction head,—about 17 feet, so that the entire available head, 270 head is utilized. An air drum is attached to each motor. The bearings of the large water wheels have considerable dimensions when the speed of rotation is considered, which fact has led to the arrangement of a device for oiling them as follows ; A reservoir is placed at the top of the receiver at considerable elevation above the bearings. Piping leads the oil to the bearings on which it is fed through bib-cocks in a continuous stream, the idea being to accomplish refrigeration as well as lubrication. The babbit of the bearings is grooved to facilitate the spread of the oil and to facilitate its flow as well as its distribution. Passing from the bearings the oil is delivered into pipes leading to and submerged in the cold water of the tail race, returning to a pump which lifts the cool oil to the reservoirs to be used again on the bearings. The temperature of the water in the tail race is essentially constant and not above 45° Fahrenheit.



SNOQUALMIE REGULATOR AND WATER MOTOR

Regulator.—The water regulation is also upon original lines. On top of the motor is bolted a rectangular cast iron box 6 feet high, the top of which opens into the elbow from the receiver and the bottom opens into the motor. A jacket of boiler iron surrounds the regulator box with neat cement packed in between. A steel pin through the middle of the box near the top of it, supports two heavy cast iron leaves pendant from it, making a hinge-like device, fitting the case closely nearly to its base, and adjusted to swing apart and together again according as the pressure of water is greater on one side or the other. The lower ends of the two opposite leaves incline inward and present a curvature that permits of sliding back and forth on a half moon base. The water passing over the outside of the leaves to the motor is independent of the water enclosed by the leaves, the latter having its source independently, but under the same pressure, from a separate connection with the receiver. Because of the bending of the leaves, their outside area is less than their inside area, therefore, although the same pressure per square inch exists on both sides of the leaves, there is a greater gross pressure on the inside, which causes them to fly apart, and so restrain the flow of water. A butterfly valve arrangement in the independent 6 inch connection with the receiver, automatically regulates the pressure proportionately to the load, so that the supply of water to the motor is regulated according to the varying loads. This device is very sensitive and quick acting.

Generators.—There are four generators from the shops of the Westinghouse Electric and Manufacturing Company. Each one complete, weighs about 100,000 pounds and stands fourteen feet high.

Generators are of the revolving armature type and deliver a three-phase current at 1,000 volts, 7,200 alternations. Normal full load current is 1,000 amperes per conductor. The armature winding consists of 266 bars with one bar per slot, and is a closed circuit winding. The armatures are 96 inches in diameter and weigh approximately 24,000 pounds each. The speed is 300 revolutions per minute and the peripheral velocity is accordingly nearly a mile and a half a minute.

Massive collector rings of the ventilated type deliver current to the external circuits. Three brushes bear on each ring, and to insure equal division of current between them in case of unequal contact resistance, separate cable leads of considerable length connect brushes and the outside circuit in order that the fixed resistance with each brush may be large compared with the possible variable resistance. The field frame is split vertically and rests on the bed plate which supports the armature bearings. Poles are laminated and cast in the field frame. The field winding is of one layer copper strap bent cold on edge and afterward insulated. At each end of a coil are brass brackets which rigidly hold it on the pole piece, exposing both inside and outside surfaces to air circulation to carry away heat. This form of construction produces a coil that is very easily insulated, furnishes a maximum surface for ventilation, and at any time may be easily repaired in case of accident. At no load these generators require a field current of 95 amperes at about 90 volts, and with full non-inductive load, 100 amperes to maintain the same E.M.F.

Speed regulation in a water power plant depends greatly on the kinetic energy in the moving parts, which in this case is ample. In these armatures about 4,500,000 foot pounds is stored at 300 R.P.M., and from the construction of the water motors, the moving water column also contributes to the stored energy, since it is rigidly connected to the revolving parts of the water motor. A hasty inspection might lead one to believe the energy in the water column was large compared with that in the generators on account of its immense weight. However, this is not the case. The penstock and receiver contain about 1,200,000 pounds of water, and with one water motor running has only 67,000 foot pounds of kinetic energy, or about one and one-half per cent. of that in a generator armature. When all the water motors are in operation the water contains much more kinetic energy, but it is still of little importance compared with that stored in the revolving armatures. Again, since two generators are geared together mechanically, the stored energy in both will be available to prevent sudden changes of speed in either, due to changes in load, and the speed regulation therefore still further improved. The stored energy in one of the revolving armatures is equal to the electrical output of that armature at full load in four seconds.

Exciters.—Two separate 125 volt exciters of 75 K.W. each are provided for supplying field currents to the generators. They are driven by separate Snoqualmie motors. No speed regulators are used since the load is perfectly steady. Regulation is effected by hand.

Switchboard.—The switchboard is constructed in panels of pure white marble and presents a dazzling appearance with its mountings of brass and bronze and its complete assortment of switches and instruments for measuring voltage, amperage and watts. There is probably not a switchboard west of the Mississippi river as elaborate as this one. It is 35 ft. 5 in. long and 7½ ft. high.

Current is conducted by lead covered cables laid in sewer pipe in the cement floor, from the generators to bare aluminum bus bars extending along the south wall to the switchboard. These bars are supported by brackets on glass insulators. They are of pure aluminum one-fifth inch by three inches in section and in thirty foot lengths. Three bars carry 1000 amperes with a very moderate rise of temperature. Joints are lapped and bolted and connection to switchboard is made by cables with brass terminals, which are bolted to the bars. Each generator panel is provided with two circuit breakers, three load ammeters, a field ammeter, two watt meters of the Niagara type, synchronizing lamps, three pole generator switch, field circuit plug switches and a field rheostat. The watt meters are arranged by a simple combination of series and shunt transformers, so that their indication is the same as if they were on a two-phase circuit. This is of considerable value, as the indication of the meters remains equal with a balanced load even if the power factor on the three-phase circuit is less than one.

Transformer House and Transformers.—The fire-proof house is made of brick and iron with a concrete floor with a ground area of 40 x 60 feet, and 30 feet high and stands just east of and contiguous to the intake. In this building, the current is received at an initial voltage of 1000 and is then passed into a series of step-up transformers where the voltage is raised to 30,000, which is nominally the voltage of transmission.

One Hundred and Fifty Mile Circuit.—An ingenious and elaborate high tension plug board has been constructed in this transformer house overhead by the use of which any combination of transformers and circuits can be arranged. There are similar plug boards at Renton, Seattle and Tacoma, and it will be possible by the use of these plug boards conjointly to arrange a combination so that a circuit may be made from the Falls to Seattle and back to the Falls, then to Tacoma and back to the Falls again, a distance of 151 miles. There are twelve 500 K.W. transformers in the transformer house, arranged in two rows longitudinally, with an aisle space on each side and a 1000 volt wire space between. All circuits entering this building are supplied with lightning arresters for the protection of the works, although lightning in this country comes at intervals of only three or four years.

Raising transformers are in delta connection for both primary and secondary circuits. Separate primary feeder panels are provided for each transformer by means of which any transformer may be controlled at the switchboard. Each transformer panel is provided with a circuit breaker, a double pole double throw washer switch and an ammeter. Voltmeters for the entire board are supported on a swinging arm at one end. Groups of three feeder circuits have each a polyphase integrating watt-meter of the Westinghouse type. Leading up the shaft to the transformer house, are 24 bare aluminum conductors, carried on ordinary glass cable insulators on the timber frame work built in the shaft. Raising transformers are of the standard Westinghouse self-cooling oil insulated type, and have a capacity of 500 K. W. each at 90 per cent. power factor. Their cases are 55 inches by 72 inches and 66 inches high, and contain 500 gallons of oil. The primary winding is for 1000 volts and the secondary winding is for either 15,000 or 30,000 volts. Each complete transformer weighs approximately 10,850 pounds, of which 3,600 pounds are in iron and copper. The most remarkable feature about these transformers is their low self-induction. With one of these windings short circuited less than three per cent. of the normal E.M.F. at 7,200 alternations per minute will send full load current through the other coil. This is due to subdividing and closely sandwiching the primary and secondary coils, which are thin and flat.

High tension coils have many layers and but few turns per layer and each layer is wound in the same direction to reduce the difference of potential between the successive layers. At the end of each layer the wire is carried across the face of the coil to the starting side. Low tension coils are of bars or straps wound on edge and afterwards insulated. Coils are spread at the ends outside of the iron core to facilitate oil circulation which carries away heat, and also to increase the distance between the coils where it is difficult to apply solid insulation. To protect attendants and apparatus in case of accidental contact between high and low tension windings suitably adjusted spark gaps are connected between each low tension winding and the earth. Each transformer is supplied with two high tension fuse-circuit breakers by which it may be disconnected either by hand or automatically in case of excessive current load. The fuse-circuit breaker consists of two hinged wooden rods by which the fuse terminals are widely separated, breaking the arc when the fuse is ruptured by the current. The electric arc at 30,000 volts is six feet long.

Lightning Arresters.—Protecting each incoming line wire is a Wurts lightning arrester of improved construction. On the front of a vertical slab 24 inches wide and 65 inches high are mounted the spark gaps cylinders in units of seven in separate porcelain cases, while on the back of the same panel are six choke coils mounted on three marble wings which effectually support the coils and insulate them while they are brought in close inductive action with each other. The whole is simple, self-contained and fire-proof.

Wire-Outlets.—Insulated wall bushings are provided for each high tension wire, consisting of a marble slab 24 inches square set into the wall, through the center of which passes a thick glass tube 24 inches long and 2 inches in outside diameter which surrounds the wire. The wires are led out from a cupola on the ridge of the roof occupying about a third of the length of the building.

The Cottages and Park.—Comfortable and commodious cottages with all modern conveniences have been erected by the company near the works, for the homes of the electrical attendants. The Company has also provided and furnished an attractive cottage for the use of the executive officers and entertainment of the Company's guests. Twenty-seven acres of ground covering the works, Falls and a considerable stretch of the river have been parked and laid out in lawns, walks and flower beds, which together with the trout fishing and hunting near at hand and the mountain vistas makes the spot one of the most attractive in the State of Washington. The grounds are illuminated by arc lamps.

HIGH TENSION TRANSMISSION SYSTEM.

Right of Way.—The right of way for the pole lines is owned by the Company with the exception of several stretches where the county roads are occupied under a franchise. The right of way is patrolled daily by men on horseback, each patrolman having a distance of ten miles, and reporting by telephone to Seattle and the Falls from the telephone booths located every three miles. This patrol service has been organized for the purpose of detecting any weakness that may appear in the line, and to protect it from any dangers that may threaten. The private right of way of the Company is in general fifty feet wide. The right, however, was obtained and was exercised, by which the company in a great many places for a distance of 300 feet on each side the line has the privilege of felling timber which might injure its operations. A great many thousand dollars were spent in this way, which the casual observer might think unnecessary. Sound trees eight or ten feet in diameter, three hundred feet in height, have been cut as far back as 300 feet from the line, assuming that if they fell for any reason they might reach the line and damage it. No falling tree can possibly reach the pole lines.

Aluminum Lines.—The circuits are of aluminum and this is the first instance of its use in the history of long distance transmission. The conductivity of this metal is about 60% of copper, so that the wire in order to have a capacity equal to copper must be about 60% larger, but with this increased size the weight is slightly less than one half as much. Longer spans are therefore possible, thus making a saving in poles and insulators. Aluminum is non-corrosive in a desirable degree. It has great tensile strength. Because of its larger cross section its radiating surface is larger and the wires keep cooler. These advantages are apparent in aluminum conductors in addition to the fact that they cost less than copper.

Poles.—The poles are all cedar, 9 inches at the top, stripped of their bark and either burned or tarred at their butt end. The standard length is 36 feet, set to a depth of six to eight feet, but they vary from this to a length of 150 feet according to the nature of the country. All poles have been set in line with a transit and their lengths are conformed to the regular grades established by the engineer's levels.

Seattle and Tacoma are reached by separate pole-lines paralleling each other forty feet apart as far as Renton, 19 miles, and there they diverge to their respective terminals,—Seattle 31 miles, Tacoma 44 miles. The country transversed is varied in topography from the mountainous and rough to the rolling and flat, and a great portion of it is covered with the primeval forest.

Aluminum tie wires are also used, and soft No. 8 B. and S. guage is found best suited to the work. No electrolysis can occur with aluminum ties, as would be the case with iron or copper.

Wire Splices.—Line conductors are spliced with the sleeve joint, which consists of a flattened aluminum tube 9 inches long and with walls one-sixteenth of an inch thick, large enough to snugly enclose two wires. The whole is given three complete twists by special clamping tools, when the joint is complete.

The Seattle lines contain about 67,000 pounds of wire and the Tacoma lines 72,000. The charging current on one of the Tacoma lines on open circuit at 30,000 volts is 7.4 amperes, which is small compared with the load current, so that no trouble is experienced from static capacity. The lines are of aluminum approximately .26 inches in diameter for the Seattle circuits and .23 inches in diameter for the Tacoma circuits, and are designed to deliver respectively 4,000 and 2,000 kilowatts at 25,000 volt pressure and 80 per cent. power factor.

Measurements made on the Seattle line show a resistance of 84.5 to 88 ohms, depending on the temperature, and self induction at 7,200 alternations, equivalent to 54 ohms, for a circuit composed of any two wires. The resistance of any two wires of the Tacoma circuit measures 152 ohms. These results show that the performance of the line is as it was designed.

Insulators.—The lines are carried on imperial porcelain insulators, Redlands type, which are four and one half inches high and six and one half inches in diameter, and which weigh four pounds each. Parafined locust pins are used, which support the insulators four inches above the cross arms. Two circuits are run on each pole line, one on each side, with a triangular spacing of 30 inches between wires. Fonr wires on the lower cross arm are spaced on either side 25 inches and 75 inches from the center of the pole. Twenty-five and a half inches above on another cross arm are two wires 40 inches on either side of the pole.

Cross-Arms.—Cross-arms are four and a half by six inches, by 10 and 8 feet long. The whole line is built in the most substantial manner. Poles are braced whenever necessary, and on all curves and turns and crossings double cross-arms are used. The length of span on the Seattle line varies from 90 to 150 feet, but the average is 110 feet. On the Tacoma line the average span is 150 feet. About 15 inches sag is allowed, which is considerably greater than is common practice with copper wires.

Transpositions.—Transpositions divide the lines into six equal sections on either side of the Renton sub-station. The spans in which transpositions are made are between poles set six feet apart, which effectually prevents accidental contact between wires. At each transposition the circuits are given one third of a turn, always in the same direction.

Telephone System.—A telephone line of No. 10 B. and S. aluminum wire is carried about five feet below the power circuits on ordinary glass insulators on brackets. It is transposed at every fifth pole. Its service has been very satisfactory. Patrol men carry portable telephones so that a call can be made at any point by climbing a pole. Telephone booths containing also line supplies and tools, are located every three miles.

Issaquah Sub-Station.—Ten miles from the Falls the pole lines run through the town of Issaquah, a coal mining camp of about 1200 people. At this point a small brick station has been erected, from which to distribute power to the coal mines and for general lighting in the town. The station serves also as a residence for the patrolman on that beat.

Renton Sub-Station.—Renton is a prosperous village of 2000 people, is the center of a large farming district and the seat of extensive coal mines which will likely soon be operated electrically from this plant. At the Renton station a high tension switchboard similar to that at the generating station, allows any combination of the incoming and outgoing circuits to be made. Each wire is provided with a high tension fuse switch. All high tension apparatus and power apparatus at this station is contained in a handsome and commodious brick building 40 x 40 feet and 22 feet high inside, which was designed and erected for this purpose. High tension wires entering this building have the marble and glass bushings similar to those at the generating station. This station is also the home of the patrolman's family having charge of this station.

Kent and Auburn.—The Tacoma circuits pass through the towns of Kent and Auburn, villages of about 1000 people each, supported by a prosperous farming and dairy communities. Auburn is the junction point of the Northern Pacific Railway, where the roads from Seattle and Tacoma meet the overland system. The Power Company has in contemplation the establishment of small sub-stations at these places for local distribution.

Terminal Sub-Stations.—The Seattle building, Corner of Main Street and 2nd Avenue, is substantially built of Tenino sand-stone and iron. The machinery occupies the underground basement, while the Company offices and renting tenants occupy the street floor space and above. The Tacoma Station is a large red brick structure and is occupied jointly with the Tacoma Railway & Power Company.

Seattle Station.—The Seattle sub-station is in the business district, and the high tension wires are carried to it on 70 foot poles, painted blue and black. A tower on the third floor about 40 feet above the ground receives the wires, which are thence led through a terra cotta enclosure to the lightning arresters on the second floor. These are six in number and similar to those at the Falls station except that they are for 25,000 volts instead of 30,000 volts. Below, on the ground floor, are high tension fuse-switches by which the incoming current may be controlled. All of the load may be placed on either transmission circuit, or it may be divided so that the steady load on one circuit and the fluctuating load on the other, or both circuits may be connected in multiple. Here, as elsewhere, is shown the extreme flexibility of the switching apparatus, which is a characteristic feature of this plant. In this room are also high tension fuse switches for the lowering transformers immediately below in the basement.

Step-Down Transformers.—Of the lowering static transformers there are at present three, with primaries and secondaries in delta connection, for supplying 500 volt direct current rotary transformers. Primary coils may be connected either for 12,500 volts or 25,000 volts. Secondary coils have intermediate terminals by which the E.M.F. supplied to the rotaries may be adjusted when necessary. Otherwise the transformers are similar in construction and capacity to those of the generating station. There are also three pairs of two to three phase lowering transformers of 300 K.W. capacity each with 2,000 volt secondaries for general lighting and power distribution, two induction motors, which makes the total station transformer capacity 2,400 K.W.

Switchboards.—Elaborate white marble switchboards are furnished for controlling the three rotary transformers. The alternating current panels each contain a three pole single throw washer switch, three ammeters, synchronizing lamps, field rheostat and a switch for the starting motor on the rotary. On each direct current panel is a circuit breaker, ammeter, volt-meter, plug receptacles and two single pole washer switches. The feeder panels have circuit breakers, single pole washer switches and Thomson recording watt meters. Volt meters are mounted at the end of the board on a swinging arm.

Rotary Transformers.—There are two rotaries of 500 K.W. capacity delivering current at 550 volts which are operated in multiple on both alternating and direct current sides. Having eighteen poles the speed is 400 revolutions per minute. Collector rings and brushes are like those on the generators but of smaller capacity. To bring the rotaries up to the synchronism without excessive starting current, a small induction motor is mounted on the armature shaft. As it has sixteen poles it is enabled to bring the rotary armature slightly above synchronism, but the final speed adjustment is made by varying the field current which imposes more or less load from iron loss in the rotary armature. Several cross connections in the armature windings insure superior mechanical and electrical performance.

Alternating 2,000 V. Current.—For alternating current lighting and motor work a panel is provided for each set of three-phase, two-phase transformers, containing two 2,000 volt automatic circuit breakers, two ammeters, two Niagara indicating watt meters, two double pole throw switches and pilot lamps. The feeder panels are the same, except polyphase integrating watt meters of the Westinghouse type replace the indicating watt meters. In making changes on these boards the circuits are first opened by the circuit breakers, as the switches are not designed to break current.

CONCLUSION.

The function of this Company will be to furnish power for all purposes and it is expected that every wheel turning in Seattle and

Tacoma will very soon be propelled by the power from Snoqualmie Falls. Negotiations are on foot also for the establishment of electrolytic and chemical works, and it is not unlikely that within a year large blocks of power of this description will be taken which will necessitate increasing the capacity of the plant. Power for the electric lighting system and railways of both towns is furnished by this Company, as well as all the small and larger motors for the different services. The Centennial Flour Mill Company in Seattle became the first customer of this Company nearly a year before the project was completed. Nearly the entire output of power was taken before the company turned a wheel.

Efficiency of Plant as a Whole.—Calling the energy of the water at the Falls unity, the percentage thereof available at the time of full load at different points in the system will be as follows :

Energy of water	1.00	1.00
“ from water wheel	0.80	0.80
“ “ generator	0.75	0.75
“ “ step-up transformers	0.72	0.72
“ to step-down transformers	0.61	0.65
“ from step-down transformers	0.59	0.63
“ “ rotary transformers	0.53	0.57

The first column is computed for a line loss of 15%. It is intended to increase the amount of wire on the line so that this loss will be reduced to 10%.

This Company has been fortunate in developing its project at a time when most of the steam plants in its territory were worn out and ready to be renewed, and at a time when prosperity has begun to stimulate Seattle and Tacoma with new growth and vigor.

EXTENSIONS.

The design of the plant contemplates doubling the present capacity which from the present outlook appears will be necessary within a year. It will be remembered that the shaft will accommodate another penstock of the same capacity ; the intake has been built for double this plant, and so has the tailrace. The poles will accommodate twice as many wires ; the right of way has been bought and improved sufficient for any expansion, and the sub-stations will accommodate machinery for several times the initial power. Another cavity and additional machinery and wires are the only extensions necessary to double the plant. Still other cavities might be built, the shaft and tunnel enlarged, dams built in the mountains, until the entire flow of the river is absorbed, if the demand for power at the industrial centers, or a future manufacturing town at Snoqualmie Falls, ever justified it. This is cited only to suggest the future possibilities of the enterprise.

Cheap Power.—The Power Company does not engage in the distribution of power to small customers, but sells power in large blocks for this purpose to the Seattle Cataract Company, and the Seattle Electric Company in Seattle, and the Tacoma Cataract Company and Tacoma Railway and Power Company in Tacoma, reserving for itself the duty of supplying large plants of every description which can be reached by any of its lines or extensions thereof. Although this company controls the only water power commercially available to the Sound cities the Company's policy has been to build up a business of large volume at moderate prices, rather than small volume at high prices. The natural conditions make this power one of the cheapest of development in the United States, and therefore the Company is able to offer lower rates than in most other sections of the country, and special rates for nine months power. The Pacific slope is rich in raw materials and the market for finished product extends from Alaska and Mexico to Japan, China and the Phillipines. An empire has sprung up upon the western border with a heritage so vast and resourceful as to claim the attention of the civilized world.

TESTS OF BRICK MASONRY PIERS.

E. J. MCCAUSTLAND, M.C.E., '97.

It is a very well known fact, that in all forms of brick masonry construction, the crushing strength developed under loading is but a small percentage of the strength developed by the individual brick when tested singly between unyielding steel plates. It is usual to account for this wide discrepancy by saying that the excess of resistance developed by the single brick is due to the fact that the bearing plates in contact with its surface prevent lateral motion and thus hold the brick up to a higher duty. This explanation is, no doubt, in a measure true; but that it should be sufficient to fully account for the wide difference in developed strength seems, to say the least, open to question. It is the purpose of this paper to briefly consider this question in the light of numerous experiments on brick piers.

In the reports on the extensive experiments made at the Watertown Arsenal on the strength of brick piers, especial emphasis is laid upon the importance of securing a good quality of mortar in order to insure a high efficiency of the masonry. The argument is as follows: (See Watertown Reports, 1884, page 74.)

(1) "In examining the results of the cement and mortar tests, superior strength is found in the Portland Cement; likewise the piers laid in Portland Cement give the highest resistance, while piers of the same kind of bricks, but laid in weaker mortar, showed lower results, from which it appears that the quality of the mortar employed is a very important function in the strength of the brick-work."

(2) "A material tested between cushions weaker than itself suffers by the disintegration of these cushions."

(3) "The mortar acting as a cushion to distribute the load of compression, the nearer the physical properties of the mortar approach those of the bricks, the more strength will be developed in the piers."

The tables embodying the results of the Watertown tests, and published in their reports, seem to fully bear out the first (1) conclusion. As to exactly *what* quality is most desirable in the mortar, does not yet seem to be definitely settled as will be pointed out more specifically later on.

As to conclusions (2) and (3), a comparison of the efficiencies of a number of piers may be made by a study of the following Table.

These efficiencies are based upon the strength of single bricks and the crushing strength of six inch cubes of the component mortar. The last column gives ratios of compression of 6″ cubes under load of 4,000 pounds the neat Portland Cement being taken as 1.00.

The table is compiled from the report of tests for 1884, and only those are taken which are nearly 72 inches in length, the purpose being to eliminate as far as possible the effect of variations in strength due to varying length.

TABLE I.

WATERTOWN SERIES, 1884. DIMENSIONS OF CROSS-SECTION, 12″ x 12″.

No.	Height.	Composition of Mortar.	Strength, lbs. per sq. inch.			Effici'nci's of pier.		Compression of 6″ cubes of mort. under 4000 lbs.
			Mortar.	Single brick.	Pier.	% of Mortar.	% sing. br'k.	
290	72″	Neat Portland cement	3480	11406	2375	68	20.8	1.00
294	72″	1 Portland cement, 2 sand	548	11406	1792	320	15.7	1.67
300	73″	1 lime, 3 sand, 1 Portland cement	192	11406	1411	735	12.4	4.80
301	73″	1 lime, 3 sand, 1 Rosendale cement	183	11406	1646	900	14.4	3.00
293	72″	1 Rosendale cement, 2 sand	162	11406	1972	1210	17.3	5.00
288	72″	1 lime, 3 sand	124	11406	1133	910	9.9	12.00

The piers chosen for comparison are of practically equal dimensions, and they are arranged in the table in order of the crushing strength of the mortar as tested in the form of six inch cubes. The decrease in the efficiency of the pier when referred to a single brick, from the neat Portland Cement to the lime mortar, is quite marked and uniform if exception be made of the piers in which Rosendale cement is used. Likewise, the increase in efficiency when referred to the strength of the mortar, is just as marked and uniform with the same exception. Considered alone, the Rosendale mixtures show results entirely unlike the others. A decrease in the crushing strength of the mortar shows an increased efficiency when referred to either standard. These results do not seem to bear out conclusion (3) as quoted above. If the amount of compression of a six inch cube of neat Portland Cement under a load of 4,000 pounds be taken as standard, the compressions of the Rosendale mortars are respectively three and five times as great, yet the efficiencies of the piers referred to a single brick are as $1 : \frac{2}{3} : \frac{6}{7}$ a decrease in strength not at all commensurate with the increased compressibility. It is to be noted also that the efficiencies when referred to the mortars, are approximrtely as 1 : 13 : 18.

It is quite evident that the quality to be sought in a good mortar for the brick-work is something different from, or at least something in addition to, mere compressive strength; for while in the above tests the mortar composed of one part Portland Cement and two parts sand, had a compressive strength of more than three times that of the mortar composed of one part Rosendale and two parts sand, yet the efficiency of the pier as a whole was increased only 3½ per cent. It is more than probable that the tensile strength of the mortar is of more importance than its compressive strength, and that the adhesive power of the mortar is really the decisive element in its value. It is to be noted that a lime mortar of proportions 1 : 3 gives fairly good results although its crushing strength is low. The Watertown tests show that when compared to a mortar of Portland Cement and sand of proportions 1 : 2 although the ratio of crushing strength is about 4 : 18 the ratio of the pier efficiencies is about 4 : 7. The method of failure in all piers tested has been quite similar; a longitudinal cracking and pulling apart of "the bricks in one course opposite the end joints of the bricks in the adjacent course." It would seem therefore that in a mortar for brick-work, high tensile strength and strong adhesive power are the qualities to be assiduously sought.

The Watertown tests show a greater strength in piers built with the brick placed on edge, and it was argued that this increased strength came from a decrease in the number of joints, and the consequent reduction in volume of mortar used. It is more probable that the increase is due to the tendency of the bricks to act as beams when the bearings are uneven, and under these conditions the edgewise brick has an advantage over a flat one.

It is quite evident, that up to the present time, no set of experiments has been planned and carried out which would fully take account of all the conditions which might affect the strength of a brick pier: viz:—

(1) The fineness, specific gravity and age of cement used.

(2) Kind of sand, size and uniformity of grain, etc.

(3) Amount of water used in mortar.

(4) Crushing and compressive tests of single brick.

(5) Tensile tests of brick. Practically nothing has been done in this line.

(6) Crushing and Compressive tests of mortar.

(7) Tensile tests of mortar.

(8) Tests to determine adhesive power of mortar. This is a field in which much may be done. The extremely wide variations in the results of tests for the adhesive strength of mortars is indicated by the table compiled in 1888 by Emil Kuichling and quoted on page 94 of Baker's "Masonry Construction."

(9) Crushing and compressive tests of masonry.

In the "Transactions" for 1898, there was reported a series of tests on brick piers built in the Laboratories of the College of Civil Engineering. The results there reported are not fully comparable to the Watertown tests, as the crushing strength and compression of the mortar were not separately determined. The efficiency of these piers when referred to a single brick is much higher than the efficiency reported from Watertown, but the crushing strength of the single brick is much lower. This difference is doubtless due to the fact that the mortar adhered more strongly to the medium Newfield brick than it did to the harder New England brick. The mortar used was uniform throughout the whole number of piers, but the length was variable. However the influence of varying length was not at all marked.

In the Summer of 1898 another series of brick piers, fourteen in number, was built in the College Laboratories. The purpose of this series was to determine, if possible, the value of various devices used to secure a bond in the masonry in addition to the ordinary mortar joints. In order to attain to this end the piers were constructed as follows: Section 13 x 13 inches and about 80″ high, the bond showing a whole and a half brick on each face, the center being filled with a half brick. Each pier was made up of 30 courses, the joints averaging about 0.30 inches in thickness.

The mortar used was composed of Portland cement and sand in the proportions of 1 : 2. Two piers were constructed in which this mortar only was used in the joints. Two had two iron straps 1 ¼ x ⅛ inches in cross section placed in the horizontal joints at every fourth course, alternating at right angles, and secured by pins or keys which were brought up to a bearing against the outside of the pier. One pier had similar straps in every sixth, and another in every eighth course.

In two piers, a wire netting of ½ inch mesh, and wire of 0.045 inches diameter was placed in every second course, carefully bedded in the mortar and covering the entire cross-section of the pier. The same kind of netting was placed in two other piers under similar conditions except that the netting was placed in every horizontal joint. Finally, four piers were built, in which, at every fourth course, was placed galvanized iron plates 0.026 inches thick. The effect of these plates was to separate the pier into a number of distinct short masonry blocks.

The following table gives a condensed summary of the record of the tests as to crushing strength and the efficiencies based upon the strength of a single brick. The compression was not measured on all the piers, and that subject is discussed in another place.

TABLE II.

No.	Dimensions, 13″x13″ by	NATURE OF JOINTS.	Strength, lbs. per sq. in.	Efficiency, per cent. of single brick.
1	78 inches	Cement mortar (1:2)	1150	(average of 3), 30
2	78 "	" " "	1184	
Series of 1897.	92 "	" " (av. of 3)	921	
3	81 "	Iron straps every 4th course	810	(average of 2), 24
4	81 "	" " " " "	920	
5	80 "	" " " 6th course	780	22
6	81 "	" " " 8th course	843	24
7	79 "	Wire netting every 2nd course	1136	(average of 2), 33
8	80 "	" " " " "	1248	
9	79 "	" " every course	1692	(average of 2), 46
10	80 "	" " " "	1440	
11	80 "	Iron plate every 4th course	1030	(average of 4), 28
12	79 "	" " " " "	1145	
13	80 "	" " " " "	972	
14	80 "	" " " " "	858	

Tabulating these results in the order of the efficiencies, and comparing with the clear mortar as standard, we have:

TABLE III.

NATURE OF JOINT.	Efficiency, compared to pier with mortar joints.
Piers with netting in every joint	1.53
Piers with netting in every second joint	1.10
Piers with clear mortar joints	1.00
Piers with plates every fourth joint	0.93
Piers with straps every fourth joint	0.80
Piers with straps every sixth joint	0.73
Piers with straps every eighth joint	0.80

The number of these experiments is altogether too few to furnish a basis for even the most general statement; however, when taken in connection with the more extended Watertown tests it is clear that they show a marked tendency to bear out the contention that adhesive power and tensile strength are the prime requisites for a first class mortar.

The increase in efficiency of the piers containing the netting, and the decrease in the piers containing the plates and straps is exactly what might have been predicted from a study of the Watertown tests.

Both the netting and the plates seemed to control the method of failure of the pier to a great extent, as there was a marked absence of the longitudinal splitting which is the persistent characteristic of the pier constructed with clear mortar joints. In nearly every case the netting was broken by tension in the direction of both sets of wires, but in only one case (Pier 11) were the plates affected.

Tests for compression of the masonry were made on six of the piers and the results are tabulated below. Two micrometer rods were used, each 56 inches long and were placed on opposite sides of the pier. The average of the simultaneous readings was taken to compute the amount of shortening. In all cases, the initial load on the pier was 2000 pounds, or about 12 pounds per square inch. In order not to disturb the plaster of paris cushion between the pier and cross-head when the pressure was released, the load was not reduced below the initial point, but it was reduced to the initial reading after each reading of the micrometers.

The moduli of elasticity were determined by plotting unit stresses as ordinates, and relative compressions as abscissae for each pier; then, an equalizing line was drawn through the origin, and the tangent of the angle of inclination of this line to the horizontal was taken as the value of *E* for that pier.

TABLE IV.

No. of pier.	NATURE OF JOINT.	Compression in inches in measured length of 56 inches under load of				Moduli of elasticity, lbs. per sq. in.
		60000 lbs.	100000 lbs.	150000 lbs.	175000 lbs.	
8	Wire netting every 2d course------	0.0256	0.0417	0.0677	------	994,000
9	Wire netting every course ---------	0.0195	0.0323	0.0518	0.0595	1,040,000
10	Wire netting every course ---------	0.0214	0.0379	0.0585	0.0705	960,000
14	Iron plates every 4th course-----	0.0245	0.0418	-------	------	830,000
13	Iron plates every 4th course-----	0.0282	0.0475	0.0782	0.0910	680,000
5	Iron straps every 6th course-----	0.0248	0.0452	0.0890	------	783,000

These results show that the shortening of the piers constructed with wire in the joints, is less than in the case of piers with plates or straps. This is, perhaps, caused by the fact that the latter do not prevent the

TABLE V.

SHOWING RESULTS OF TESTS ON BRICK PIERS. COLLEGE OF CIVIL ENGINEERING, CORNELL UNIVERSITY.

Crushing strength of single brick 3525 lbs. to the inch. Dimensions 13″x13″x80″. Thickness of joints 0.30″. No. of courses 30. Age when tested about 7 months.

No.	KIND OF JOINT.	Ultimate Stress.		Efficiency per cent. of single brick.	Compression in inches in measured length of 56″ under load of,			Moduli of Elasticity.
		Total.	lbs. per sq. in.		60,000 lbs.	100,000 lbs.	150,000 lbs.	
1	Portland cement mortar (1 : 2)	194,000	1150	30				
2	Portland cement mortar (1 : 2)	200,000	1184					
Series of 1897.	Portland cement mortar (1 : 2)	Average of 3 tests.	921					
3	Iron straps every 4th course	136,400	810	24				
4	Iron straps every 4th course	155,400	920					
5	Iron straps every 6th course	130,000	780	22	0.0248	0.0452	0.0890	783,000
6	Iron straps every 8th course	142,500	843	24				
7	Wire netting every 2nd course	192,000	1136	33				
8	Wire netting every 2nd course	208,000	1248		0.0256	0.0417	0.0677	994,000
9	Wire netting every course	282,000	1692	46	0.0195	0.0323	0.0518	1,040,000
10	Wire netting every course	240,000	1440		0.0214	0.0379	0.0585	960,000
11	Iron plate every 4th course	174,000	1030	28				
12	Iron plate every 4th course	193,500	1145					
13	Iron plate every 4th course	162,000	972		0.0282	0.0475	0.0782	680,000
14	Iron plate every 4th course	143,000	858		0.0245	0.0418		830,000

tendency of the pier to "bulge" or increase in cross-section under stress. It is to be noted that the term "compression" as used in this paper really means "shortening" and that there could be no true compression unless the pier was laterally restricted. This condition is rudely approximated in the case of the joints of netting and hence the decrease in shortening is partially accounted for. Whether or not this restriction is sufficient to fully account for the difference can not be predicated on the number of tests made. It would be comparatively easy in future tests to give attention to this factor.

The foregoing Table (Table V.) gives a condensed summary of the records of the tests, and shows the comparative shortening, efficiencies, and moduli of elasticity.

Preparations are now being made for the construction of an additional series of brick piers, to be tested in the College laboratories. In their construction especial care will be exercised to determine definitely everything that may be of interest or value with reference to the nature, quality or condition of the material used. It is hoped that the results may be of special value.

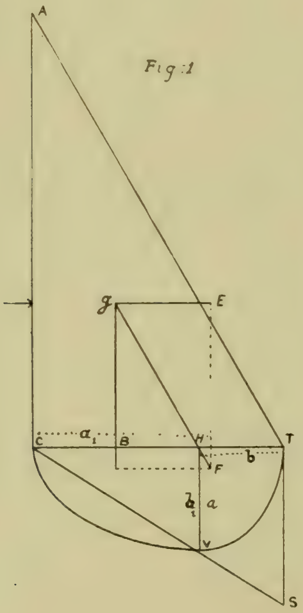
INVESTIGATION OF THE DISTRIBUTION OF PRESSURE ON THE BASE OF DAMS, BY TESTS MADE ON AN ELASTIC MODEL.*

F. F. HALL, '99.

INTRODUCTION.

The attention of the writer was first called to this matter by the article of Mr. Francis in Vol. 19 of the Transactions of the American Society of Civil Engineers.

He there gives his reason for believing a large masonry mass to be elastic rather than perfectly rigid, the assumption on which masonry dams are designed. Because of this yielding of the material the pressure on the base is more evenly distributed, and the point of maximum pressure is thrown back farther toward the center of the base line. Mr. Francis claims, "the fundamental principle to be, that different parts of the mass yield unequally to pressure, depending on the opportunity to yield laterally to the direction of the pressure, the effect being to transfer pressure from the parts yielding more easily to compression to those yielding less easily."



According to the deductions from experiments on hand-specimens, the pressure would be mainly transferred from the down-stream toe, where it is a maximum theoretically, to the interior of the base, where the resistance due to pressure is much greater due to lateral support.

The usual theoretical distribution of pressure is as shown in Fig. 1 by the ordinates to the line *C S* while the distribution according to the above theory would be in accordance with the ordinates to the curve *C V T*; the area between it and the line *C T* being equal to the area of the triangle *C T S*. (In this figure it is assumed that the resultant pressure cuts the base at the extremity of the "middle third.")

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*Abstract of Thesis by F. F. Hall, Cornell '99.

Mr. Francis says that he is unable to give the exact form of the curve, but that some of its characteristics are :—the ordinates at *C* and *T* must be small ; the maximum ordinate must be less than *TS*, and be either within or near the middle third of the base. The curve *CVT* has these characteristics and is composed of the quadrants *CV* and *VT* of two ellipses, the common ordinate *HV* at the intersection of the resultant *gF* with the base *CT* being 0.6366, *TS* being taken as unity.

It will be understood, that the curve *CVT* is given only as an illustration of the distribution of the pressure according to the theory advanced, and with no pretension to precision, either as to the distribution of pressure or the determination of the point where the pressure is a maximum.

The object of this experiment or test is then to determine the character of this curve of pressure and its peculiar features. By character is not meant its exact form, or the derivation of any equation for its accurate determination, but merely the general governing principles and proportions.

DESCRIPTION OF APPARATUS.

Many different methods of measuring the pressure on the base of the model and also of producing the water pressure or its equivalent on the face were considered ; and some of them worked out, until an unsurmountable obstacle or a better method presented itself.

The apparatus for measuring the pressure on base as finally constructed is shown in Figs. 2, 3, 4. Fig. 3 also shows the method of applying pressure to the face of the model. This part of the apparatus will be described later.

The base on which the model rested was composed of boards 1″ x 3″ x 4′ 6″ glued together into a solid piece 12″ x 3″ in section so as to prevent the warping that would be liable to occur in a plank of the same section. The upper surface of this base (on which the model rested) was carefully dressed to a true plane surface. The size of model as first intended, and according to which this apparatus was designed, was base 21.6″ x 8″, height 25.8″.

In the top of this base-board 23″ from one end, a slot 1″ wide was cut across at right angles, into which fitted the key (*A*) having a vertical play, and just prevented from coming in contact with the sides of the slot by ball-bearings as shown at *B* (Figs. 2 and 3) running between hard wood facings.

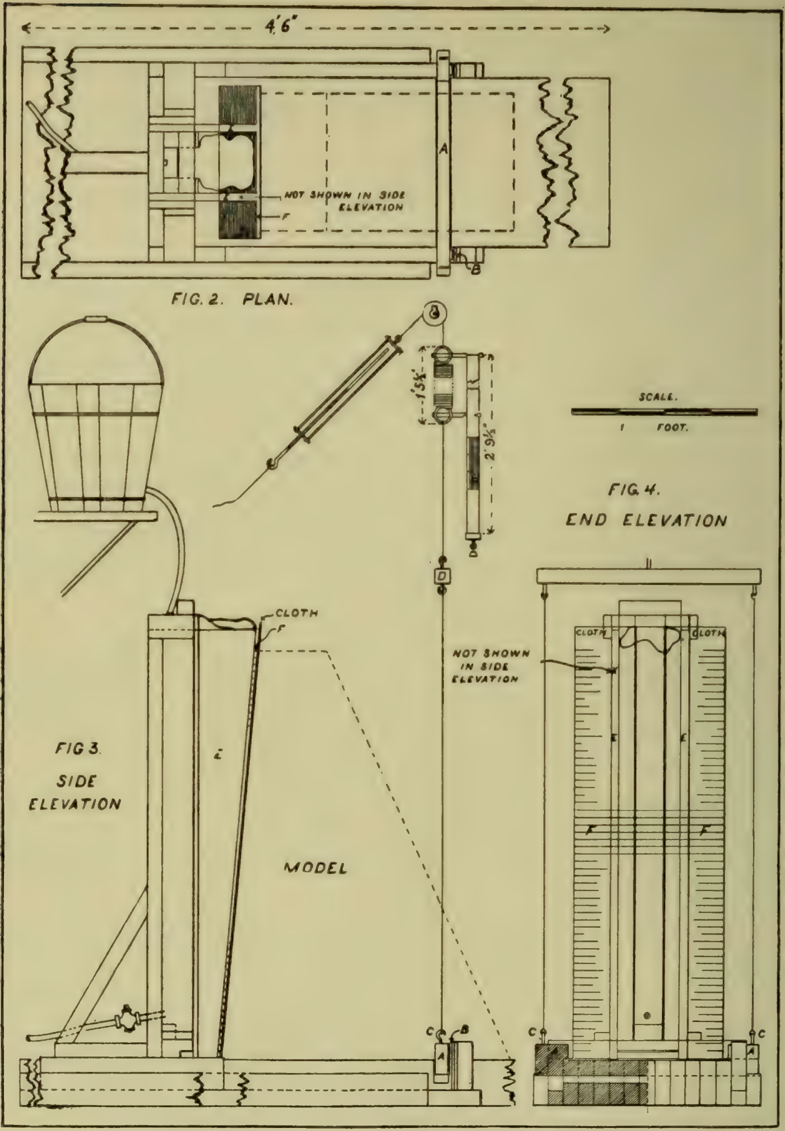

FIG. 2. PLAN.

FIG 3. SIDE ELEVATION

FIG. 4. END ELEVATION

These bearings were only placed on the side of the key toward the down-stream toe of model, as the water pressure would cause the model to shove the key in that direction. The balls extended equal distances, above and below the plane of the base-board, so as to prevent any tendency of the key to twist and bind in the slot. Attached to the ends of this key by means of screw-eyes (*C*, Fig. 3) were two strings about three feet long extending up to a strip of wood (*D*) parallel to the key below. (At first stiff wire rods were used, but they were given up for flexible strings as they caused the key to turn and bind in the slot, unless kept perfectly vertical.)

The strip (*D*) was suspended from its middle by a string running up to the spiral spring as shown in Fig. 3. The weight of model was calculated in advance to be between 95 and 100 lbs. from the known cubic contents, and the specific gravity of the material, as determined from a sample. If this pressure were evenly distributed along the base, the pressure on the key, wherever the model should be placed on the base-board, would be equal to the weight of the model ÷ 21.6, or 4.12 lbs. If the resultant pressure of the water and weight of model does not pass outside the middle third of the base, the maximum pressure that could be caused would be twice the average, or 9.25 lbs. The spring must then be prepared to measure about 10 lbs. For safety, a spring was ordered that would stand a pull of 15 lbs. without injury, and giving an elongation per pound of not less than 1″. Its total length when elongated by a 15 lb. pull was not to exceed three feet. This limit in length was fixed by the head-room allowable for setting up the apparatus.

After much trouble in obtaining a spring made as ordered, and after many delays, a suitable tempered steel spring was finally obtained from the Miller & Van Winkle Co., manufacturers of steel wires and steel springs, of Brooklyn, N. Y. Dimensions of this spring are given in Fig. 3. It was carefully calibrated by the writer so that it could be read directly to tenths of a pound and estimated to hundredths. Two brass arms were soldered to either end of the spring as shown ; the upper one acting as a support to a paper scale kept straight by a weight attached to its bottom, and the lower arm acting as a pointer for reading scale and guide for it to slide in. Due to the manner of attaching these arms, it was found that their angle with the axis of the spring was altered if the string attached to the rings at the ends of the spring were shifted even a trifling amount along these rings ; and thus a variation in the reading of the scale was produced. To prevent this, a short piece of flexible copper wire was soldered to each ring and the strings attached to these wires ; the spring being then recalibrated.

The string supporting this spring ran over a pulley in the ceiling vertically over the center of the key (*A*) and thence to a cleat at a convenient point near at hand. Between the pulley and the cleat the string was broken and a turn-buckle inserted for making the final, slow and careful adjustment before taking the reading of the spring. Needle points were attached to the base-board on either side of the slot and when these points came in a line with a mark made on the key (*A*) the upper surface of the key would be exactly in a plane with the top of base-board, and the spring, if read, when this adjustment was made would give the pressure on the base of the model at the point under which the key might be placed. By shifting the model along the base-board any portion of the base might be made to come over the key and the pressure at that point determined.

The apparatus finally adopted for producing approximately the equivalent water pressure on the model consisted (see Fig. 3) of a vertical trough mounted on runners, sliding in guides at the sides of the base-board. The face of this trough toward the model was made of rubber sheet as shown by heavy lines in Figs. 2, 3, and 4. This sheet was confined on the sides by a sliding board (*E*) whose edges were cut to such a curve (determined by trial) as to just come in contact with the face of the model when distorted by the water pressure, and thus prevent the rubber from bulging out between model and side-boards. As the ideal water pressure (computed later) was less than that of water, the intensity of pressure of the water against model was reduced by inserting a number of thin slats glued to a piece of unbleached muslin, (as shown at *F*, in Figs. 2, 3, 4) between the rubber sheet and the model. The width of the rubber sheet limited by the boards, *E*, in contact with these slats was less than the width of the model, and thus the intensity of the water-pressure on the face of the model was reduced, since the pressure of the water through the medium of the slats was distributed over a greater area. The slats formed a sheet flexible in a vertical direction and rigid in a horizontal. In making a measurement, the model was first placed in position on the base-board, and the trough then run up and pressed against the model (the slats first having been placed between model and trough) until the curve of the sideboards was nearly in contact with face of model from top to bottom. Water was then let into the trough through a rubber tube, as shown, until it reached the height of the model. The whole trough was then moved forward again until the sideboards would just miss making contact with slats, but close enough to them to prevent the rubber from bulging out between. It was necessary to clamp the trough in this position as the pressure of the water reacting against the face of model would have caused the trough to slide backward and destroy the adjustment. The height of the water in the trough was kept constantly at a level with the top of the model. The key was then adjusted and the spring read ; after which the water was drawn off from the trough, the model shifted, and the whole operation repeated for the new position.

COMPOSITION OF THE MATERIAL OF WHICH THE MODEL WAS MADE.

It was required that this material should be perfectly elastic, that it should give a distortion capable of measurement by the apparatus used and yet be sufficiently firm to stand alone and support the pressure on its face. The material that was chosen seemed to be the best at command that could be cast into the required shape and fill the above requirements. This material is that used in making printers' rollers.

After trial of many formulae, the following was finally adopted, as the most satisfactory :—

Glue	2 parts by weight.
Molasses (black)	4 " " "
Water	5 " " "

These were mixed, allowed to stand till all water was absorbed, then cooked two hours.

DESIGN AND CASTING OF MODEL.

The original intention of the writer had been to use a model of the same section as that of the proposed Quaker Bridge Dam, as being generally admitted to be a modern design of the best description. The mould for this model was made of wood, which, however, warped and leaked when the hot fluid was poured into it, thus spoiling the first casting.

A new mould was made of tin with all soldering on the outside. This mould was then encased in a wooden frame to give rigidity. On further consideration of the matter of the section to be adopted for this new model, it seemed best, that for the purpose of this experiment, a triangular or trapezoidal section would be better, as the theoretical and actual distributions of pressure could be more easily traced and more satisfactorily compared. The section was designed so that the resultant pressure would pass through the extremity of the middle third of the base, thus giving a distribution of pressure over the whole base. The dimensions were as follows : (Fig. 5) $h' = 25''$, $b'' = 2''$, $b' = 15''$, and thickness $= 8''$; the mould being built up several inches higher to allow for shrinkage of material in cooling, this surplus to be cut off after hardening. The writer found that previous experiments with the samples tried, in which the time of cooking was accurately noted, were valueless when applied to large boilers in which cooking took place more slowly. As the material required several hours to set, the method of experimenting as the cooking progressed was impracticable. The only means at command was the unnatural and unsatisfactory one of pouring a small tin-full out of the large boilers, and placing the tin on ice; when the material would harden in a few minutes, but lose firmness on warming up, and did not give a fair indication of what the ultimate firmness would be. The material was cooked three hours, when it was thought to be right, but, on removing the casting after two days, it lacked sufficient firmness to stand upright. The model was allowed to lie on its side for five days, as previous samples had hardened considerably in the first few days after making, but it still refused to stand, though of the same firmness as a sample having the same proportionate dimensions that would stand. This was owing to the greater pressure on the base of the large model, due to its own weight.

As both time and means at the command of a student for his thesis work are limited, it became necessary to make this model serve, or throw up the experiment. The model was therefore cut down to a smaller size with a proportionately larger base. These new dimensions were (Fig. 5) $b'' = 1''$, $h' = 15''$, $b' = 11''$, width $= 6\frac{1}{2}.''$ This gives the cubic contents as 585 cu. in., and the computed weight using the heaviness of .045 lbs. per cu. in. (from specific gravity of sample) as 26.325 lbs. which checked with the actual weight. These dimensions give a trapezoidal distribution of theoretical pressure, the bases of the trapezoid being nearer equal than would have been desirable, (see later computations for theoretical distribution of pressure) but a shorter base was to be deprecated as that would have increased the ratio of the width of the key to the length of the base; whereas it was desirable to have this ratio as small as possible. It was also necessary not to make the length of base too small in proportion to the height of model in order to have the material stand upright.

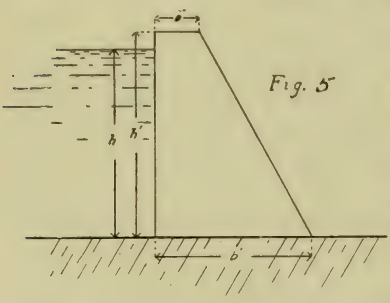
Fig. 5

DESCRIPTION OF METHOD OF MAKING EXPERIMENTS.

The model was placed on the base-board of the apparatus so that the sides of the base came in contact with two parallel lines drawn on the surface of the base-board from end to end, equidistant from its center and 6½″ apart (the width of base of model). The model was moved along between these lines, until the slot was just under the upstream toe of model, and yet entirely covered by model. The apparatus for producing water pressure on model was then adjusted, and the pressure applied as previously described.

TABLE A.

Number.	SPRING READING IN POUNDS.			Ordinates to actual distribution curve in inches ½ lb. = 1″.	Ordinates by calculation to theoretic trapezoid in inches ½ lb. = 1″.	Ordinates to ellipses obtained by calculation from the formula for ellipse, semi-axes being given.
	Friction against Spring.	Friction with Spring.	Average.			
1	1.25	0.96	1.005	2.01	3.1381	2.246
2	1.55	1.45	1.500	3.00	3.3028	3.118
3	1.88	1.80	1.840	3.68	3.4677	3.747
4	2.14	2.06	2.100	4.20	3.6325	4.240
5	2.44	2.34	2.390	4.78	3.7974	4.642
6	2.57	2.50	2.535	5.07	3.9622	4.975
7	2.69	2.63	2.660	5.32	4.1270	5.253
8	2.78	2.68	2.720	5.44	4.2919	5.483
9	2.93	2.83	2.880	5.76	4.4567	5.671
10	2.97	2.89	2.930	5.86	4.6215	5.821
11	3.02	2.94	2.980	5.96	4.7864	5.937
12	3.02	2.96	2.990	5.98	4.9512	6.021
13	3.05	3.00	3.025	6.05	5.1160	6.072
14	3.07	3.01	3.040	6.08	5.2808	6.088
15	3.05	2.98	3.015	6.03	5.4457	6.062
16	2.97	2.90	2.935	5.87	5.6105	5.930
17	2.96	2.88	2.920	5.84	5.7753	5.688
18	3.76	2.66	2.710	5.42	5.9402	5.322
19	2.45	2.37	2.410	4.82	6.1050	4.804
20	2.05	1.99	2.020	4.04	6.2698	4.075
21	1.39	1.32	1.355	2.71	6.4346	2.985

The key was then raised by pulling on the string, running over the pulley in ceiling and down to the spring, until the scratches on the end of the key came nearly in line with the needle point, care being taken not to pass these lines. The string was then fastened to the cleat, and the key raised slowly by means of the turn-buckle until the scratches came exactly in line with the needle point, when the spring was read and noted in table (see results in Table A) opposite reading No. 1 and in the column headed "Friction Against Spring." The key was then raised by means of the turn-buckle until it pressed up into the base of the model for an appreciable distance, when the key was again slowly lowered by means of the turn-buckle, until the scratches were once more in line with the needle points. The spring reading again noted and entered in table opposite reading No. 1, and in the column headed "Friction with Spring." The water was then drawn off from the apparatus for producing the water pressure, which was then unclamped and drawn back, and the model moved back ½ the width of the key. This was determined by lines drawn parallel to the edge of key on the base-board at the required intervals, the upstream edge of the base of model being brought successively in contact to these lines and the spring reading taken in each position as described above. In this way 21 readings were obtained.

DISCUSSION OF RESULTS AND THE CHARACTER OF THE CURVE OBTAINED.

In plotting the curve of pressure (see Fig. 6) the axis or base was taken 11 in. long (full size), and the force scale of ½ lb. to 1 in. adopted for plotting the ordinates. The ordinates plotted were the average of the two spring readings obtained for each position of the model, and were plotted at the position along the base line occupied by the center of the key at the time of obtaining that specific reading or ordinate.

These ordinates were thus ½″ apart (shown in Fig. 6 by vertical lines, Nos. 1 to 21, terminated by small circles). Through the extremities of the longest ordinates (Nos. 11 to 16) a smooth curve was passed then a horizontal tangent to this curve drawn, and the point of tangency noted. This would be the position of the maximum ordinate, HV, or point of maximum pressure on the base. The length of this ordinate and its distance from the left end of base line were then measured, and found to be $HV = 6.08$ in. and $HC = 7.1$ in. Assuming that the curve of pressure is composed of two quadrant ellipses with their common ordinate the maximum, as suggested by Mr. Francis, we should then have given one of the semi-axes of each ellipse equal to this maximum ordinate HV; while the sum of the other two axes is equal to the length of the base line, and the sum of the areas of the two quadrants is equal to the area of the theoretical trapezoid of pressure drawn to the same scale (½ lb. to 1 in.).

This area must represent the total weight (G) of the model or 26.325 lbs., and as the force scale is ½ lb. to 1″ this area will be 52.65 sq. in.

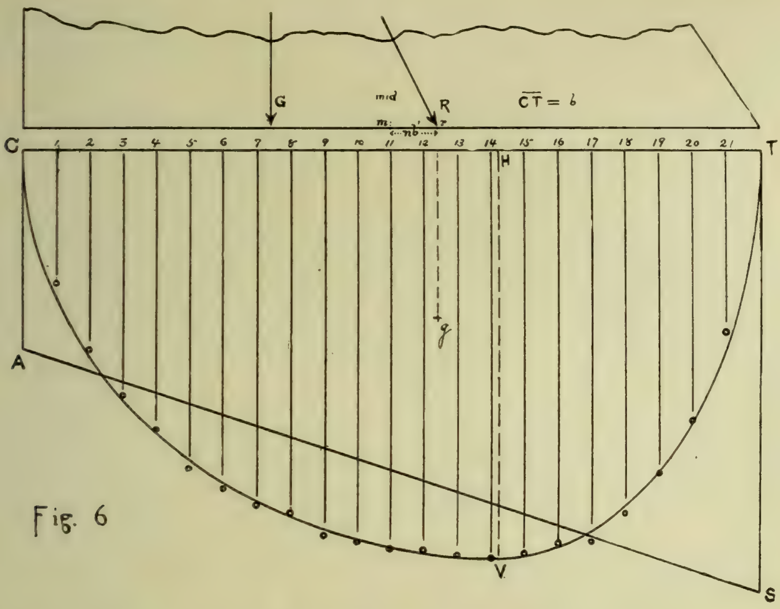

Fig. 6

Let a_1 and b_1 denote the semi-major and -minor conjugate axes of the left quadrant ellipse; (see Fig. 1) and

a and b those of the right quadrant ellipse. We then have as the area of the left quadrant ellipse, $\frac{1}{4} \pi a_1 b_1$.

For the right quadrant ellipse,

$$\text{area} \doteq \tfrac{1}{4} \pi a b.$$

Their sum $= \frac{1}{4} \pi (a_1 b_1 + ab)$;

Which must equal the area of the theoretic trapezoid or $= 52.65$ sq. in.

Then, as $b_1 = a$,

We have $\frac{1}{4} \pi a (a_1 + b) = 52.65$

And $$a = \frac{52.65 \times 4}{\pi (a_1 + b)}$$

But $(a_1 + b) = 11.$

Hence $a = 6.09414''$;

i. e., if the distribution of pressure is as Mr. Francis suggests, the maximum ordinate should be equal to 6.09414″. The measured ordinate, HV, (6.08), is then 0.1414″ too small, which little discrepancy could be easily accounted for by inaccuracies in apparatus (almost too small to be seen in plotting).

In determining this maximum ordinate (a), its exact location on the base line is not necessary in the computations, as (a) and (b) are not separately involved, but their sum ($a_1 + b$) which is known to be equal to the length of base line.

The semi-axes of these quadrant ellipses are therefore determined, viz.:

$$a_1 = 7.1'' \qquad b_1 = 6.094''$$
$$a = 6.094'' \qquad b = 3.9'';$$

and by the use of the equations to the two ellipses the ordinate for any x can be computed, and the curve was thus constructed as shown in Fig. 6. The values of those ordinates occurring at the points No. 1, 2, 3, etc., where the ordinates obtained from experiment are plotted, were determined and noted in the last column of Table A.

This curve as seen from Fig. 6, comes as near being the average mean curve passing through the small circles as any that could be drawn, except one that would pass through the last circles at either end, but this would not be as near the true curve of pressure as the one drawn, though it might be a better curve for the points located; for the following reason:—

Each ordinate as determined by experiment represents the area of a strip 1″ wide, *i. e.*, it represents the mean ordinate for that strip. Now the mean ordinate in a strip bounded by an ellipse, as in this case, occurs not at the middle, but nearer the shorter side; consequently if these ordinates at the ends of base were each shifted out towards the ends a little, until they occupied the positions of the true mean ordinates, they would come very much nearer to the curve as drawn.

DETERMINATION OF THE THEORETIC TRAPEZOID OF PRESSURE.

It was found roughly by means of graphics that the resultant pressure for the final section of model would pass through the middle third of the base, and hence n would be less than $\frac{1}{6}$.

We therefore use formulas (1a') and (2′), pages 559 and 560 of Church's Mechanics, for obtaining the accurate values of n (ratio of mr to CT in Fig. 6) and p_m (maximum pressure per unit area).

The following values were obtained:—

$n = 0.0613$.

$p_m = 0.50765$ lbs. per sq. in.

$p = \frac{G}{b'l}$ (mean unit pressure) $= 26.325/ 11 \times 6.5 = 0.36818$ lbs. per sq. in.

$p_1 = 2p - p_m$, (minimum unit pressure) $= 0.22871$ lbs. per sq. in.

The bases of the theoretical trapezoid would then be, in inches, 13 times the above, as the base of model is 6½″ wide and the force scale ½ lb. to 1″. This gives the ordinate at the heel as 6.6″ and at the toe 2.97 in. The trapezoid CAST shown in Fig. 6 was constructed accordingly.

The slope of this theoretic line of pressure was determined, and from it the values of the ordinates to the trapezoid at points No. 1, 2, 3, etc. computed (noted in next to the last column of Table A).

Taking up the characteristics of this curve, and comparing them with those stated by Mr. Francis we find that the ordinates at C and T are small, that the maximum ordinate HV is less than TS and is within the middle third of the base (in this case, however, as the theoretical resultant pressure should by design come at the extremity of the middle third, the maximum pressure HV would in all probability fall outside the middle third of the base, though near to it). The curve CVT is composed of two quadrant ellipses, CV and VT, the common ordinate HV being the maximum.

Mr. Francis does not attempt to locate the position of the maximum ordinate but suggests that it would in all probability come within, or near, the middle third, and most likely at the direct point where the theoretical resultant pressure cuts the base.

The maximum ordinate of the curve in Fig. 6 falls quite a little in advance of the point R where the theoretical resultant pressure cuts the base. This could be accounted for in two ways. First by the distortion of the section of the model (caused by the water pressure) the center of gravity of the section would be moved out toward the toe with a corresponding outward movement of the point of the theoretical resultant with the base. Secondly, the material not being perfectly elastic, but having some rigidity, the point of maximum pressure would be thrown out toward the toe to a certain extent dependent upon this amount of rigidity.

However, in order to determine this maximum pressure; provided that the distribution curve is composed of two quadrant ellipses as stated above, and as seems to be the case from the result of this experiment; it is not necessary to locate its position along the base since the formula, $a = \dfrac{G \times 4}{\pi (a_1 + b)}$ for its determination does not involve its location, its value being dependent only upon the total weight of model and width of base, varying directly with the weight and inversely with the width.

If, then, the section of a dam is designed so as to prevent overturning by giving it a factor of safety of 2 (*i. e.*, the theoretic resultant pressure must not pass outside the middle third) the crushing strength of the material will not be reached until $\frac{4}{\pi}\frac{G}{b'}$ is equal to the crushing strength of the material. The maximum pressure in this case is $\frac{2}{\pi}$ of the theoretical maximum, a ratio less than unity; hence the crushing strength of the material does not become a governing factor until a considerably greater height is reached. If it is admitted that a pile of stone will not crush from its own weight if uniformly distributed over its base when of no greater height than that reached by any engineering structure, then as $\frac{4}{\pi}\frac{G}{b'}$ is only 1.27 times G/b', (the uniform pressure) and as the location of this intensity of pressure is near the middle of the mass where it has considerable lateral support, it might possibly be permissible to disregard the crushing strength of the material entirely in the design of masonry dams.

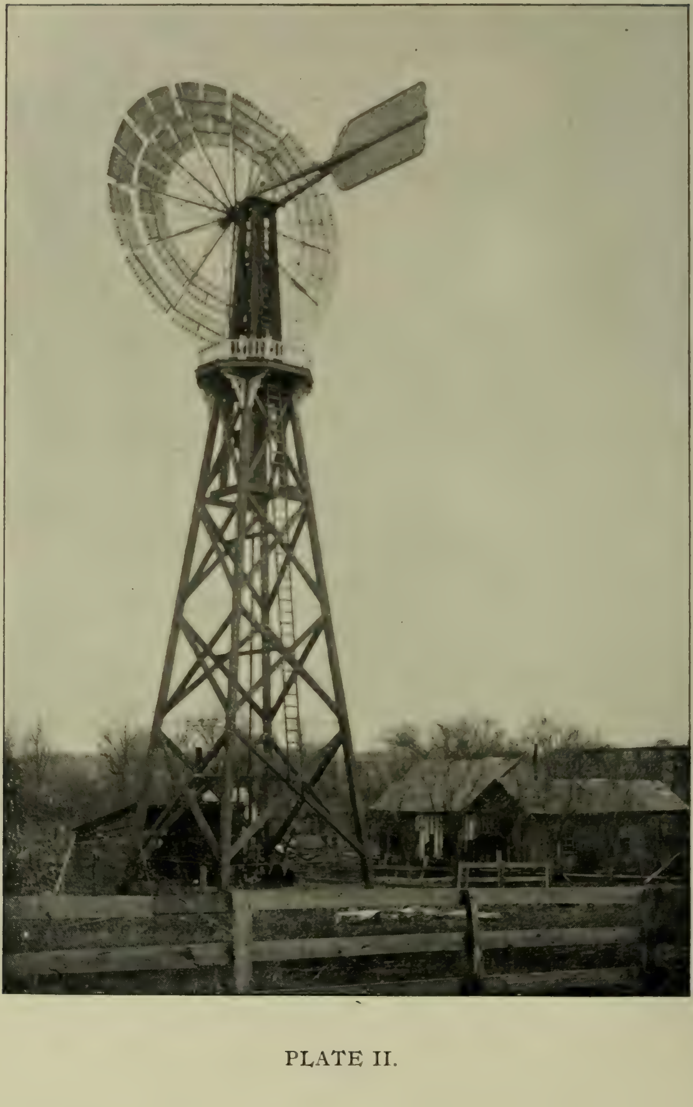

PLATE II.

WINDMILL POWER.

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Wind is the most abundant, and at the same time, most unsteady and unreliable of nature's sources of power. It is everywhere present, varying in velocity from the gentlest breeze, to the hurricane of 75 to 100 miles per hour. Its velocity is seldom uniform even for one minute. Measurements with light paper anemometers, registering each 10 seconds, show that it varies in magnitude in a few seconds by several hundred per cent. It is also constantly varying in direction.

Not only has it this gusty, unsteady, character, but for days at a time its velocity may be so small that even the most lightly loaded windmill will do no work. The power of a windmill is necessarily small. The heaviness of air is only about $\frac{1}{1000}$ of the heaviness of water, so that for a given velocity, the pressure of a moving column of air is only $\frac{1}{1000}$ that of a column of water of the same area. The area of the wind motor must therefore be very many times larger than that of the water motor for the same power.

Wind power is seldom utilized by civil engineers. The power which the ordinary windmill will furnish is so small, and so unreliable that he cannot well use it to advantage. It is better to use a more reliable source—gasoline or steam-engine—at a somewhat greater cost. The waterworks engineer in selecting a motor to pump water for a town, may, under certain conditions, find a windmill best suited to his purpose. Wind power is cheap, and where a storage tank is not too expensive—and, in the central west—where the wind velocity is greater than in the east (12 miles per hour on the average, as compared with 8 miles for the whole United States)—and where the periods of calm are short—a windmill may be used. A case of this kind is the town of Valley Falls, Kansas; a place of 1200 population; the water for which has been pumped by the windmill shown in Plate II, for several years, with the exception of two months per year, July and August, when occasionally a traction engine was employed to run the pumps.

It is the farmer who is chiefly interested in the windmill. It is to him a great labor saving machine. He can use it to pump water for stock, for sprinkling and house purposes, and if he need it, for irrigating purposes. An elevated tank will store a three or four day's supply, from whence it is distributed to watering troughs or to hydrants conveniently located not too far from the tank. It will furnish him power for shelling and grinding corn, cutting fodder, working a churn, a saw, or any small machine requiring small power. It will also furnish power to run a dynamo, which, with the aid of a storage battery, will enable him to light his home with electricity.

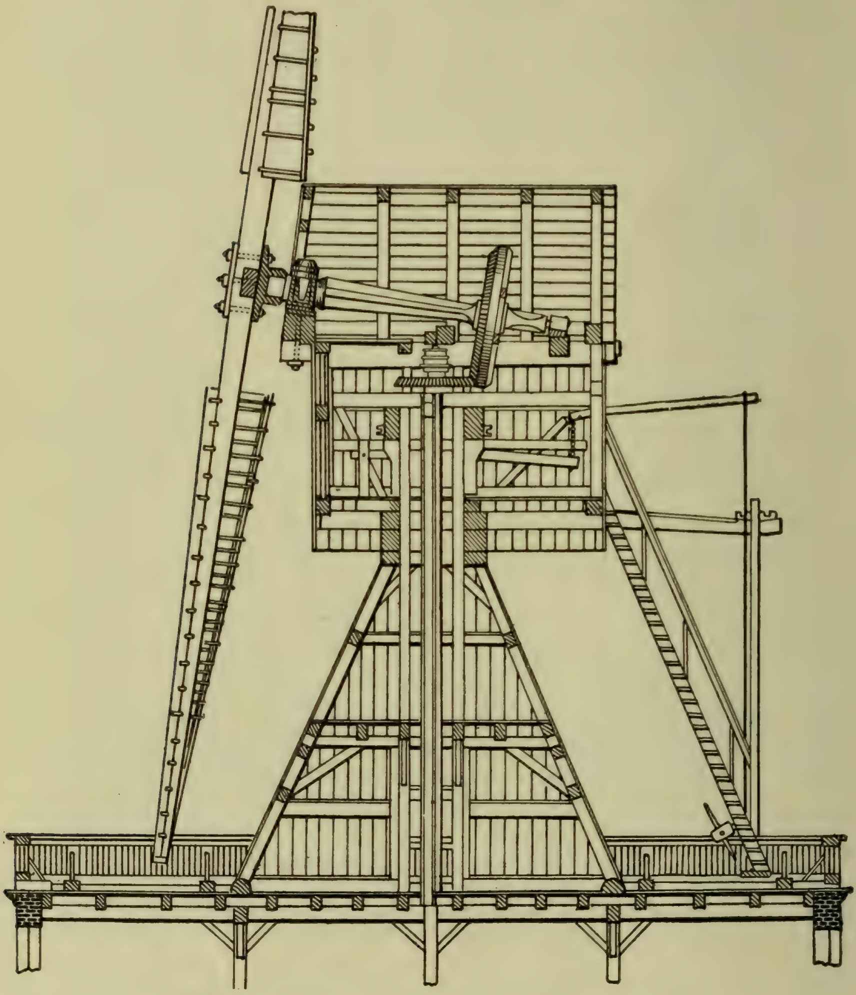


FIG. 1.—*Showing Section of Dutch Windmill.*

The European windmill differs much in appearance from the American mill. The windwheel of the former has usually four, long, wooden arms, to each of which is attached a sail, against which the wind presses. The sails consist of a frame work, on which canvas is stretched, forming usually a warped surface, the angle with the plane of the arms, called the angle of weather, being about 7° at the outer end, and 18° at the wind end. The length of sail was usually about $\frac{5}{6}$ the length of the arm ; the width at the outer end $\frac{1}{3}$ the length, and the width at the inner end $\frac{1}{5}$ the length. The sail area is seen to be small compared with the wind area, or zone containing the sails. The arms were sometimes 60 ft. long. The American wooden mill is much smaller and more compact than the European. It has six or eight arms, to which is attached a framework which carries many small sails. These sails are usually 3 or 4 ft. long, 3 or 4 inches wide at the outer and 1 to 3 inches wide at the inner end ; and set at an angle of 30° to 40° to the plane of the wheel. For large wheels these sails are arranged in two or more concentric rings. The American steel mill differs from the wooden mill principally in having larger and fewer sails for a given size of mill, having curved instead of plane sails, and offering less resistance to the passage of the air over the backs of the sails. There is seen to be a partial return in the sails of the steel mill, to those of the European mill.

Windmills are divided into two classes, pumping mills, and power or geared mills. In the pumping mill the revolving windwheel gives a reciprocating motion to a pump rod, which in the wooden mills makes a double stroke to each revolution of the wind wheel, and in the steel mill only one stroke to every 2 to 4 revolutions. In the power mill instead of the pump rod, there is a vertical rotating shaft, which gives motion to the grinder or other machine which does the useful work, and which makes six or eight revolutions to one of the windwheel, that is the pumping mill is geared back from 1 to 4, to 1, so that the wind wheel can run at best speed, and the pump not work so rapidly as to be injured, and the power mill is geared forward so as to give the grinder a sufficiently high speed to do its work to advantage.

A reciprocating pump is not a proper load for a wind motor. The work per stroke of the pump, or pump load, is nearly constant at all piston speeds. The range of pump speed is not very great, whereas the range of wind velocity is great. If the pump load is such that the pump will start in a 7 mile wind, a 15 mile wind will produce the maximum piston speed, then for all higher wind velocities the sail area must reduce itself so as not to injure the pump. The effective sail area constantly decreases as the wind velocity increases, a larger and larger amount of the sail area is wasted. If the quantity of water pumped per stroke could be increased automatically as the wind velocity increased, the pumping power of a windmill would be greatly increased. There is not this loss in the case of the power mill in grinding corn. The grinder is made in such a way that as the speed increases the amount of corn which enters the grinder increases, that is, the load automatically increases with the wind velocity. Many attempts have been made to vary the load on the pump with the wind, but they have not proved a success in practice.

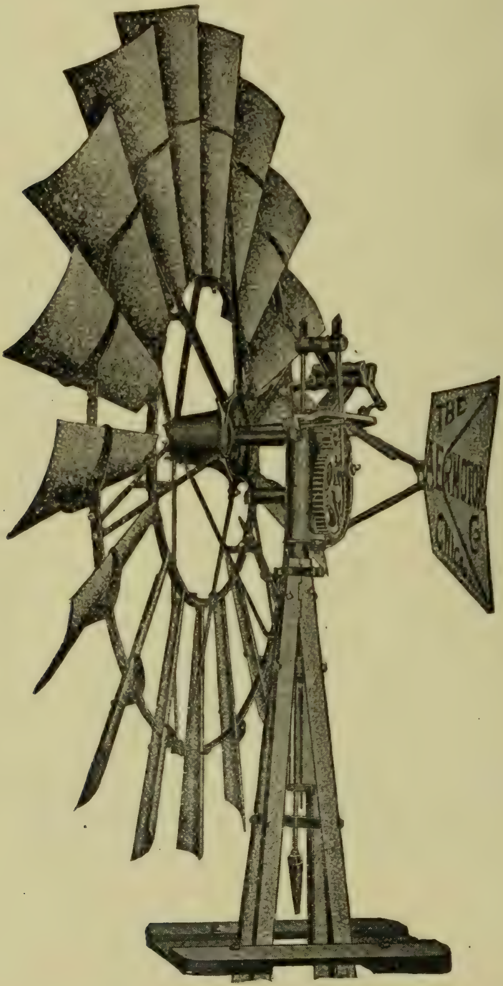


FIG. 2.—*Showing Working Parts of Pumping Aermotor.*

PLATE I.

The power of windmills has in most cases been over-estimated, or, perhaps it is better to say that the wind velocity in which a given size of mill will produce a given power has been under-estimated. This is probably due to the fact that very few tests of windmills in actual use have been made in which the wind velocity has been carefully measured with a standard anemometer. Many experiments have been made by Smeaton and Mr. Thomas O. Perry, on model windmills moved in still air. The principle being the same as rating a current meter in still water. But the conditions were quite different from those of actually working mills in gusty wind. The laws deduced by Smeaton and Perry for their small wheels carried around in still air are assumed to be true for mills in gusty wind, and this has led to an over-estimate of their power.

It is difficult to make comparative tests of windmills especially pumping mills, because the conditions cannot all be controlled by the experimenter without great expense. The pump efficiency must be determined in each case, the mills must be in equally good working order, the lift and discharge should be the same, as well as the atmospheric conditions.

The data given here are taken from the writer's experimental tests of windmills made for the U. S. Geological Survey. These tests were made principally on windmills in good working order, of various sizes and makes—scattered over various sections of Kansas. They are not comparative tests since the conditions were not the same. All the conditions affecting the working of each mill were noted, however, and by a study of these an approximate comparison can be made among them. Space will not permit the insertion of these data here, they will appear later in a government publication.

In these tests the wind velocity was measured with a Weather Bureau cup anemometer, held on a pole at the height of the axis of the wind wheel of mill, and directly in front of it, it is shown in Plate I, each half mile of wind movement, and each 100 strokes of the pump were recorded electrically on a register. From these the piston speed or number of strokes of the pump per minute for different velocities are found. From the discharge per stroke and lift or the distance from the surface of the water in the well to where it discharged from the pump, the power was found. The efficiency is the ratio of the useful work done to the energy in ft. lbs. of the cylinder of the air which strikes the sails. This efficiency is that of pump and mill combined, not of the mill alone.

TABLE I.

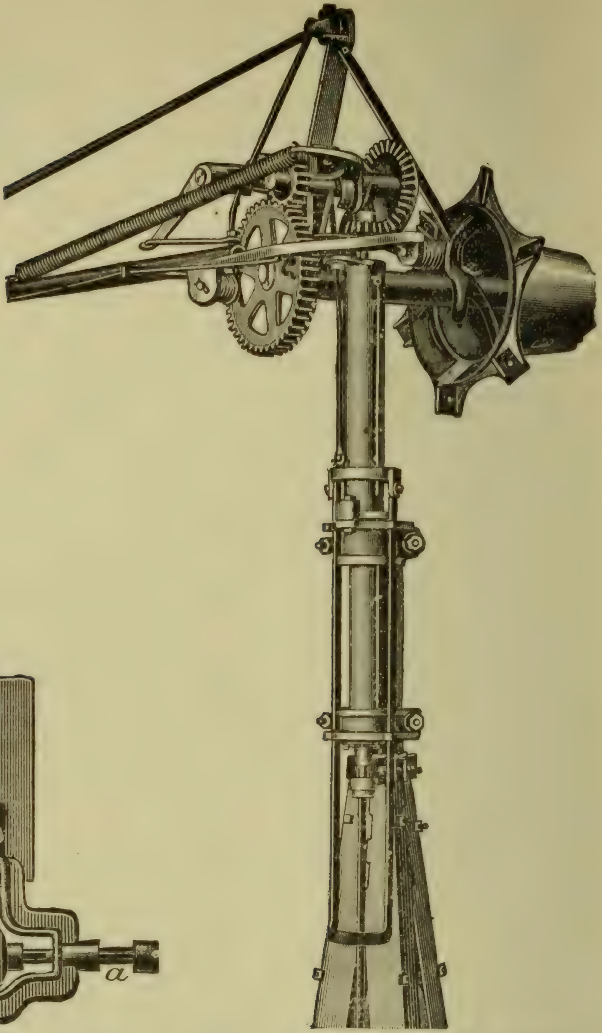
Mill No.	Name.	Size.		No. strokes per min. when wind velocity per hour in miles is						Useful work in horse power when wind velocity per hour in miles is						Ft lbs. per strok of pump.	Lift in ft.	Size of pump.
				8	12	16	20	25	30	8	12	16	20	25	30			
2	Woodmanse,	12 ft.,	Steel	----	5.2	16.0	20.3	23.3	25.3	----	.085	.260	.322	.379	.411	536	18	9⅝ x 12
3	Aermotor	"	"	5.3	12.0	16.4	19.8	23.1	25.0	.067	.151	.207	.250	.291	.315	415	14	9½ x 12
4	Ideal	8	"	----	10.2	19.3	25.3	28.1	25.0	----	.015	.029	.038	.043	.038	50	12	5⅝ x 8
6	Gem	8	"	----	12.4	17.9	22.0	25.4	28.5	----	.029	.042	.051	.059	.065	78	10	6⅛ x 8
9	Aermotor	16	"	----	10.6	14.1	17.2	19.6	21.0	----	.325	.433	.448	.601	.644	1013	44	8 x 16
11	Ideal	12	"	----	4.8	12.7	18.8	23.3	25.0	----	.123	.325	.481	.600	.639	844	45	7⅞ x 12
12	"	14	"	3.1	10.9	15.7	19.2	21.5	---	.025	.087	.125	.153	.172	----	264	11	9½ x 12
19	Gem	12	"	----	6.2	11.9	14.7	16.0	----	----	.085	.162	.201	.219	----	450	18	10 x 10
21	Halliday	12	Wood	----	14.0	28.5	37.3	44.6	----	----	.060	.121	.159	.184	----	142	15	7⅝ x 7
25	Fairbanks	8	Steel	12.3	29.2	37.3	38.0	33.3	----	.002	.005	.007	.008	.005	----	6	9	2½ x 4
36	Eclipse	22½	Wood	4.8	12.0	16.5	20.0	24.2	----	.036	.090	.124	.150	.182	----	248	39	4½ x 14
42	Ideal	6	Steel	4	13	19	23	----	----	.004	.014	.020	.024	----	----	35	17	3 x 6
48	Eclipse	30	Wood	6	15	20	24	28	----	.23	.58	.77	.92	1.07	----	1265	135	4 x 22

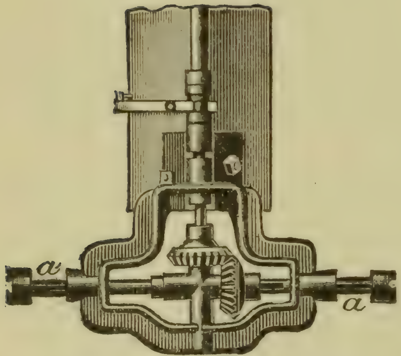
In Table I some data are given of the working of pumping mills. No. 2 is the mill shown in Plate I. No. 3 is like that shown in Fig. 2 and is lightly loaded. Nos. 21 and 48 are like the one shown in Plate II. The effect of pump load can be easily seen. No. 25 is very lightly loaded and gives little power; it pumps water for stock. No. 11 is very heavily loaded, and does the largest amount of work for high velocities of any 12 foot mill tested. No. 48 is the largest mill tested and pumps water for the town of Valley Falls, Kansas. No. 21 is a direct stroke wooden mill.

It is easily seen from the results in this table that the useful work done by the windmill and its pump in common use is small, less than ¾ of a horse power for the best 12 ft. steel mills and little more than one horse power for the large wooden mills. This useful work increases with the wind velocity up to about 20 miles per hour in the lightly loaded mills and to about 30 in the heavily loaded ones. If the mill is lightly loaded, it will start in a comparatively low wind velocity six to eight miles per hour, and will run a much larger part of the time than the mill that is heavily loaded; but it will not do as much work in velocities of 12 or more miles per hour as a mill more heavily loaded. A comparison has been made of the work that would be done by mills Nos. 3 and 11 at Dodge, Kansas, in the six months, April to September, and it was found that although No 11 worked on the average only 51 per cent. of this time, it would do 26% more useful work than No. 3 which worked 75 per cent. of the time. In the Eastern part of the United States where the wind movement is less than on the great plains, the result would be quite different. The load should be suited to the wind velocity of the locality where the mill and pump are to be used. It depends, too, on the size of the storage tank. If this tank is small, the mill must be lightly loaded so as to fill it often, if large, it can be more heavily loaded, and will furnish a large supply. The useful work done depends on the efficiency of the pump which may be anything from 20 per cent. to 80 per cent.

In the power mill this pump efficiency is absent, and the load on the mill can be easily varied, and thus the efficiency of the mill at different loads can be found. There is this difficulty, however, the wind is varying so much in amount, that it is not possible to get directly the maximum power of the mill for a given wind velocity. If, for example, the load is such as to give the maximum power in a 15 mile wind, in half a minute's time the velocity may be 20 miles per hour, or perhaps it may decrease to 10 miles, and the mill stop running. Fig. 3 shows working parts and foot gear of mill No. 27. The power was found by the use of a Prony friction brake on a pulley at (*a*) Fig. 4, on the foot gear.

Fig. 5 shows the indicated horse power for three brake loads of 2 lbs., 4 lbs., and 6 lbs. the brake arm being 35¼ inches long. The horse power is found from the formula.





FIG, 4.—*Showing Foot Gear of Aeromotor.*

FIG. 3.—*Showing Working Parts of Power Aeromotor.*

$$\text{H. P.} = \frac{2\pi l n P}{33{,}000}$$

l = length of brake arm = 35¼ inches
n = number of revolutions of brake pulley per minute.
P = load in pounds at end of brake arm
33,000 = foot pounds per minute for one H. P.

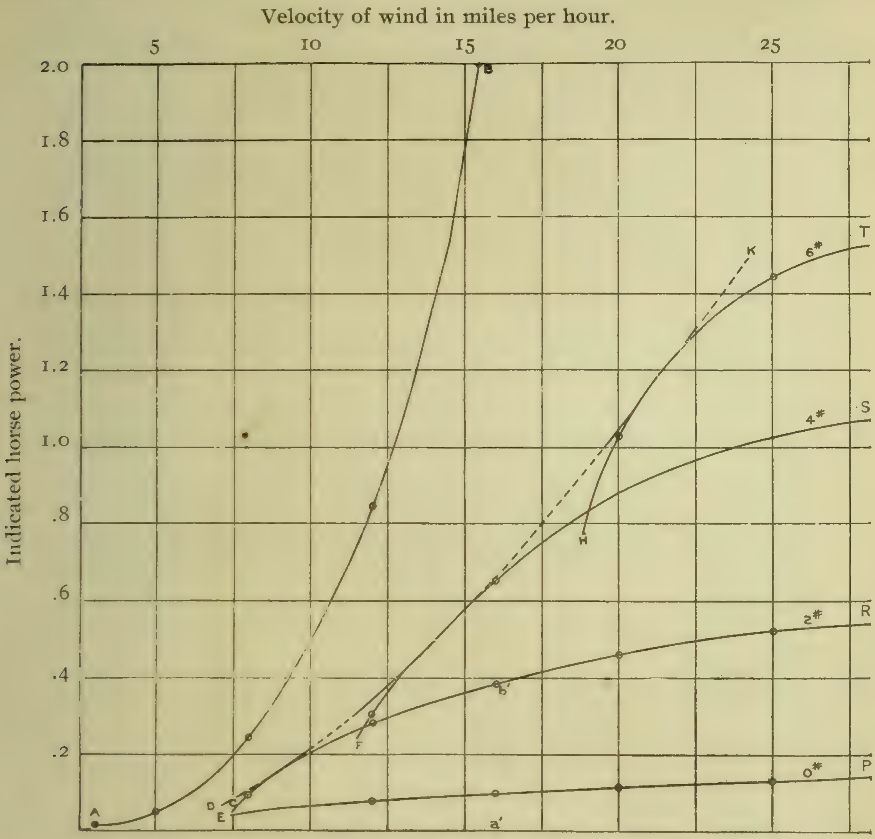


FIG. 5.—*Showing Brake Horse Power of 12 ft. Aermotor No. 27.*

The load is constant, and equal to 2 lbs. for the curve *C R*, it is 4 lbs. for the curve *F S*, and 6 lbs. for the curve *H T*. These curves are tangent to the dotted curve *D K*, which is the curve of maximum power. The curve *A B* gives the total power of the hollow cylinder of air which strikes the sails of the windwheel. The ratio of any ordinate of a load curve to the corresponding ordinate of the curve *A B* is the efficiency of the mill for that velocity. Thus in a 15 mile wind the efficiency for the 2 lb. load is 20 per cent., for the 4 lb. load it is 32 per cent. The efficiency decreases as the wind velocity increases.

The power of wooden windmills as we have found them in use in the State of Kansas, is greatly inferior to that of steel windmills of the same size. Their wind wheel speed is not as great in a given wind velocity and loading, as the steel mill. Table II shows the speed and power of one of them No. 50 which had been in use about 2 years, and was apparently in first class order, and had very good wind exposure. This table also shows the performance of two 12 ft. steel mills, Nos. 27 and 53, and that of 16 ft. steel mill No. 44.

TABLE II.

No. Mill.	Name.	Size.	No. Sails.	Weather Angle.	Sail Dimensions.	Load in ft. lbs. per revolution of wind wheel.	No revolutions per minute of wind wheel when wind velocity in miles per hour is					Horse power of mill when wind velocity in miles per hour is				
							8	12	16	20	25	8	12	16	20	25
44	Aermotor	16 ft.	18	30°	59x25¾x10½″	0	23	38	48	56	65	----	----	----	----	----
"	"	"	"	"	"	528	----	28	41	50	59	----	.45	.66	.80	.94
"	"	"	"	"	"	880	----	13	34	44	54	----	.35	.89	1.16	1.43
"	"	"	"	"	"	1408	----	----	16	36	47	----	----	.68	1.53	2.01
"	"	"	"	"	"	1936	----	----	----	25	40	----	----	----	1.47	2.31
"	"	"	"	"	"	Best	----	----	----	----	----	.15	.48	.93	1.55	2.35
27	Aermotor	12 ft.	18	31°	44x18¾x7¾	0	30	49	63	75	87	----	----	----	----	----
"	"	"	"	"	"	222	16	43	57	70	81	.089	.284	.368	.458	.523
"	"	"	"	"	"	444	----	23	48	65	77	----	.303	.653	.890	1.02
"	"	"	"	"	"	666	----	----	12	50	72	----	----	----	1.03	1.45
"	"	"	"	"	"	Best	----	----	----	----	----	.11	.35	.65	1.05	1.60
53	Ideal	12 ft.	21	32°	43¼x16½x8¼	0	25	40	51	60	68	----	----	----	----	----
"	"	"	"	"	"	272	----	29	41	51	60	----	.239	.338	.420	.500
"	"	"	"	"	"	455	----	----	32	44	54	----	----	.440	.606	.745
"	"	"	"	"	"	Best	----	----	----	----	----	.07	.24	.45	.70	1.04
50	Monitor	12 ft.	96	34°	44x4¼x1¾	0	16	33	44	54	64	----	----	----	----	----
"	"	"	"	"	"	120	----	25	35	43	52	----	.091	.127	.156	.189
"	"	"	"	"	"	314	----	----	27	34	40	----	----	.230	.324	.381
"	"	"	"	"	"	Best	----	----	----	----	----	0	.11	.23	.36	.52

Nos. 27 and 44 were new, No. 53 had been in use about 2 years, but was in very good working order. They all had good wind exposure.

The power of the wooden mill is seen to be about one half that of No. 53, and about one third that of No. 27. It is frequently stated that the power of windmills increases as the squares of the diameters of the wind wheels. Our tests do not verify this statement.

Table II gives data for two new mills. Nos. 44 and 27 of the same make, tested at the same place, one 16 ft. in diameter, the other 12 ft. in diameter. In a 20 mile wind the ratio of their power is $\frac{1.55}{105} = 1.47$. The ratio of the squares of the diameter is $\left(\frac{16}{12}\right)^2 = 1.78$. The indicated powers for these mills is as the 1.34 power of the diameters.

A 12 ft. steel power mill, and a 50 ft. steel tower weighs about 2,000 lbs. A 16 ft. steel mill and a 50 ft. tower weigh about 4,250 lbs. The weight of the 16 ft. outfit is more than twice that of the 12 ft., and its power is only about 1½ (one and one half) that of the 12 ft. In addition to this the 12 ft. governs more easily than the 16 ft. and is less likely to be injured in a storm. For most purposes, therefore, it is better to use two 12 ft mills than one 16 ft.

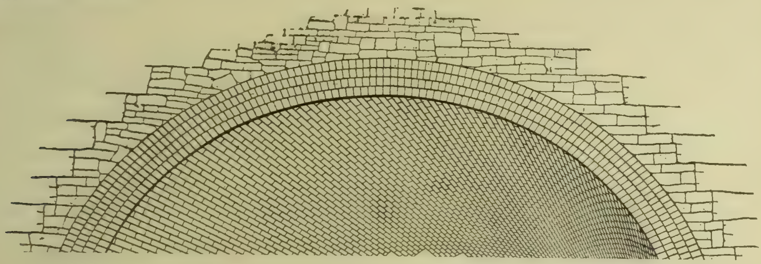
A DESCRIPTION OF THE INTERSECTION OF THE CATARACT CONSTRUCTION COMPANY'S TUNNELS AT NIAGARA FALLS, N. Y.

GEORGE F. SIMPSON, C.E., '79, M.AM. SOC. C.E.

In October, 1892, the writer was engaged by the Cataract Construction Company to design the stone work at the intersections of its power development tunnels. Before his arrival a plaster of paris model had been made, showing a proposed change of shape of the tunnels at the intersection of the tunnel from the first power house with the main tunnel. This change was not necessary nor was it desirable. In the intersections designed by the writer no change of shape was made in any of the intersecting tunnels. In the one described in this article it was necessary to furnish the working drawings in the least time practicable.

The original plan of the company was to construct a tunnel near the level of the lower river. Several mills were to be located near the head of this tunnel and the water from them was to be discharged into it. One or more inlet canals were to supply the mills with water from the upper river. This tunnel is probably the largest covered aqueduct that has ever been constructed ; it has a carrying capacity estimated at from 100 to 120 thousand horse power. This tunnel is horse-shoe shaped with the toe upward, and is lined throughout with four courses of brick. The invert is circular in section, as are all the other parts, as may be seen from the accompanying plate. The width of the invert is 14.77 feet ; its radius 26.9 feet, and therefore its versine is 0.936 feet. The side walls have soffit radii of 29 feet and their centers are on the same level as their tops. These are 10.671 feet above the top of the soffit of the invert, and the inside of the tunnel is 2.035 feet wider on each side than the top of the invert, or 18.84 feet wide. The crown arch is full-centered with a radius of 9.42 feet.

The development of electric power transmission has changed the original plan from a separate wheel pit for each mill to a system in which the power is generated in central power houses. At present there is but one mill discharging water into the main tunnel. This mill derives between six and seven thousand horse power from the water discharged. The first power house has been completed with a capacity of 50 thousand horse power and another of the same capacity is being constructed. The later power house will discharge into the head of the main tunnel.



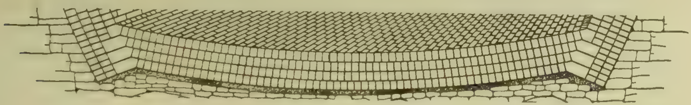
ERRATA.

P. 56. 5th line from top insert , between invert and side walls.

P. 56 10th line from bottom insert , after where.

P. 58. 3rd line from bottom change element to arris.

P. 59. On end templet change 1A2 to 2A1. Parentheses apply to 2A1.



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Intersecting Arches of Wheel Pit and Main Tunnels.

The water from the first power house is discharged into the side of the main tunnel at an angle of 60 degrees at station 68 + 73.51 from the portal. The intersecting tunnel is of the same size and shape as the main tunnel, just described. It was made so large that other wheel pits might be discharged through the bottom of this one if desirable. Its perimeter is formed by four circular curves, which make at the intersection of the soffit surfaces six quoins, each a part of a different ellipse, since the elements of the surfaces, as well as the axis, intersect the corresponding elements and axis of the main tunnel at an angle of sixty degrees. Only two of these quoins, those at the invert, are shown.

The six quoins in this intersection are all built of granite, the indi-

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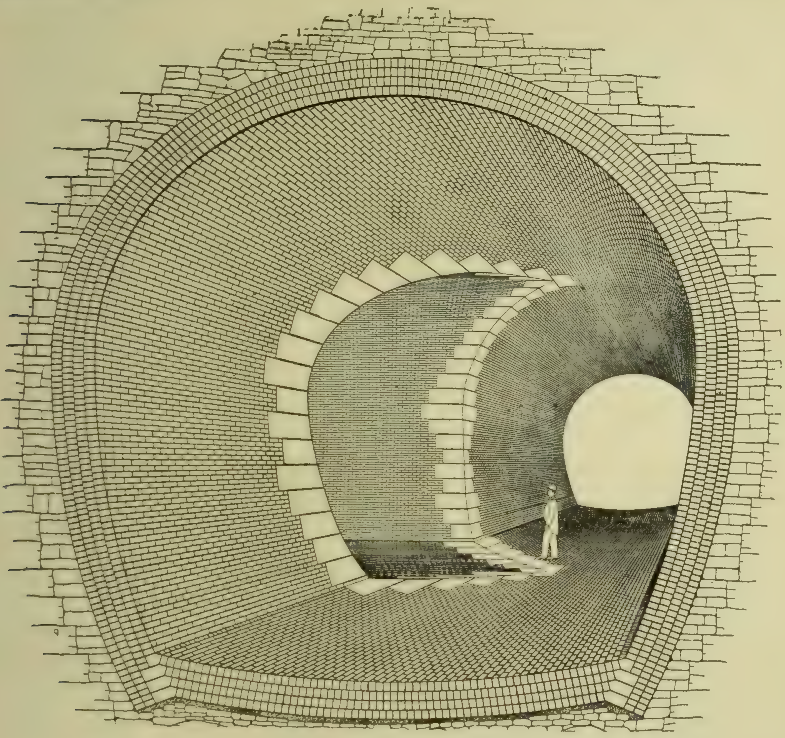

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The six quoins in this intersection are all built of granite, the individual stones of which extend along the walls of both tunnels far enough to provide for the load transmitted along the quoin and to bond with the stones above and below as well as with the brick work lining of each tunnel. Naturally the work on this intersection divides itself into three parts which the writer will name the invert side walls, and crown arch, as these divisions correspond to the corresponding parts of the tunnels. Two of the quoins are formed by the intersection of the inverts of the two tunnels, two by the intersection of the curved side walls, and two by the intersection of the full centered crown arches. These six quoins are all on one side of a vertical plane in the axis of the main tunnel, as the intersecting tunnel does not extend across to the opposite side. The two quoins in the invert meet at the intersection of the lowest element of the two inverts, forming equal altitudes in different elliptical curves having for their semi-minor axes the radius of the inverts, but having different semi-major axes. The two quoins in the crown arch meet at the intersection of the highest elements of the two crown arches, forming two different quarter ellipses having for their semi-minor axes the radius of the crown arches, but having different semi-major axes. The two quoins formed by the intersection of the side walls are equal altitudes of different elliptical curves having for their semi-minor axes the radius of the side walls, but having different semi-major axes. These quoins differ in the part of an ellipse used in the three parts of the intersection. In the invert it is the part next to the minor axes, in the side walls it is the part next to the major axis, and the part used in the crown arch is the entire quarter ellipse from the minor axis to the major axis. In all these cases the major axis is horizontal and the minor axis is vertical.

That the beds of the stones may be perpendicular to the soffit in all the quoins, they are made portions of radial planes, or planes passing through the centers of the soffit curves and parallel to the elements of the tunnels. The length of the same stone is the same in each tunnel, except some of the stones in one of the side walls, where to prevent their being too heavy, the lengths in the main tunnel and intersecting tunnel are different.

It had been decided to use brick for the lining of the tunnels near this intersection of such a thickness that nine courses would lay not less than twenty-four inches and as near that as practicable. The invert was lined with sixty-six brick courses including two special bricks at the side walls. This gave 2.71 inches for each course, or 24.39 inches for 9 courses. The side walls required 49 courses of 2.676 inches per course, or 24.084 inches for 9 courses. The crown arch required 131 courses of 2.707 inches, or 24.367 inches for 9 courses. As the average thickness of the brick was 2.39 inches; the brick joints in the invert were 0.32 inches, in the side walls, 0.286 inches, and in the crown arch, 0.3175 inches. The writer decided to make the soffit surfaces of each stone in all of the quoins equal in width to six courses of bricks in the inner ring of the tunnel lining when it could be done. This was to limit the size of the stones so they could be handled by simple machinery. This determined the width of the soffit of stones in the invert and crown arch to be 15⅞ inches, and of stones in the side walls to be 15⅝ inches, and the slight necessary variations were made in the thickness of the joints. Having fixed the width in the soffit for each stone generally, this fixed the number of stones in each quoin and the necessary changes for the peculiarly special stones. There being no two stones alike in the whole intersection, all were special stones, although a number in each quoin were similar in many details.

To this point, only the arc of the soffit covered by each stone and each joint is known, and from this arc with the formula, $\frac{\text{arc}}{R} \times \frac{180}{\pi} =$ degrees in arc, the angle subtended by each stone and each joint was calculated, R being the radius, in inches, of the soffit of the part of the work in which the stone was finally placed. The radius of the extradosal surface is twenty inches greater than the soffit radius, it being the intention to build five rings of brick in the lining of the tunnels at this intersection and near it.

To simplify the calculations and to aid in the comprehension of the work, the writer has introduced an auxiliary plane in each of the three parts of the work. These will be known as reference planes. Each passes through the axis of the portion of the cylinder under consideration; those for the invert and crown arches are vertical and those for the side walls horizontal. The grade of the tunnels is so small it was neglected in designing the intersection although the proper grade was maintained in constructing it.

The angular distance of each bed of each stone from the reference plane was calculated, and checked by comparing with the sum of the angles for all the stones and joints between the reference plane and this particular bed. In addition to the angular distance of the beds of the stones from the reference planes, the angular distance of the chief soffit elements and the soffit elements midway between the top and the bottom beds of the stones, were calculated. The angular distance of these middle elements were used later to calculate the lengths of the middle ordinates in making the templets.

The parts of this intersection were constructed in the same order as those of the brick lining, viz., side walls, crown arch, and lastly the invert. Because of the shortness of time the designing was done in the same order. In this article the natural order is followed although any part may be designed first.

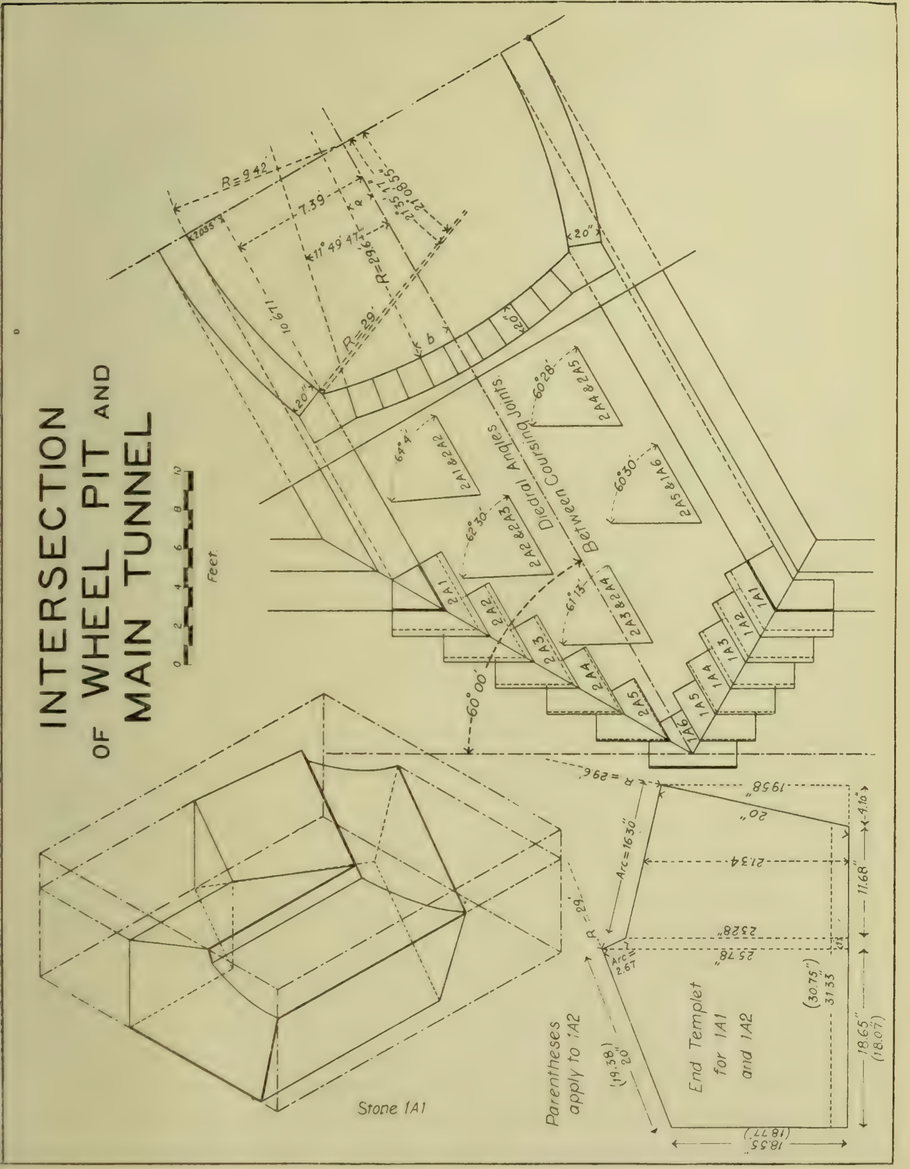
INVERT.

The invert will be considered first, recalling that the reference plane for this part is a vertical plane through the axis of the tunnel. It is not necessary to suppose a reference plane in each tunnel, as the tunnels being of the same size and shape, the results obtained for one will be the same as those for the other.

It having been decided that 15⅞ inches should be the length of arc on the soffit of a stone in the invert the other dimensions of the end templet were fixed. The arc of the extrados being 16¾ inches and sides 20 inches each. The highest brick course on each side of the invert is a special brick which runs under the side walls. These courses are 2¾ inches thick and by adding one to five ordinary brick courses we have 16.3 inches for the width of soffit of the highest stone in the invert. By letting the keystone extend three courses of brick beyond the center of the main tunnel invert, the stone joints become 0.385 inches thick.

With the formula, $R \sin \alpha = b$, the distances from the reference plane to all the arrises of the stones, as well as to the middle elements and the element between the invert and side walls, were calculated. α is the angular distance found above; and R, the radius of the soffit for soffit elements and 20 inches longer for extradosal elements. These values of b, as well as the several values of arc, α, *sin.* α, and *cos.* α, were tabulated. These values of b are useful in the construction of the invert and crown arch as in these parts the reference plane is a vertical plane through the axis of the tunnel.

Next the lengths of the arrises in the invert were found. The length of one can be assumed, and this should be the arris between the invert and side walls on the longer quoin, see plate. This can be assumed so that the stone will be long enough to reach through the side wall. But in this case it would make the stone at this point larger than desirable; hence the arris was assumed to be two feet and nine inches long. To expedite the work the corresponding element in the shorter quoin was assumed to be three feet long. As was the case in this work, the assumption of the length of two arrises may lead to a difference at the keystone, which, if desirable, may be corrected by adding the difference to the calculated lengths of all the arrises in one quoin. The bonding of the brick on the sides of the stones was required to be some multiple of four inches but not less than eight inches.



INTERSECTION OF WHEEL PIT AND MAIN TUNNEL

This was made twelve inches for each stone in the shorter quoin and thirty-two inches for each stone in the longer quoin. If $n =$ the number of joints between any two arrises, by combining the above we get

$$R \; tang. \; 30° \; (sin. \; \alpha' - sin. \; \alpha'') - n \times 12$$

inches for the shorter quoin, and

$$R \; tang. \; 60° \; (sin. \; \alpha' - sin. \; \alpha'') - n \times 32$$

inches for the longer quoin. The quantity in the parenthesis should be so taken as to be positive, α' being greater than α''. This is because we assume the highest element of the invert with which to begin the calculation. More universal formulæ would have had the double sign $\pm$ in the parenthesis and $\mp$ before the last term.

These formulæ are general but were used for the arrises of the beds *nearer* the reference plane only, and a shorter method was used to find from the length of each of these the length of the other at the same joint. The arris at the opposite side of a 0.385 inch joint in the shorter quoin will be 0.385 × tang. 30° = 0.23 inch longer at the quoin end and 12 inches shorter at the other end, or a resultant of 11.77 inches shorter. This length of 11.77 inches was subtracted from the length of the arris of one stone to obtain the length of the arris at the same joint on the next stone. The arris at the opposite side of a 0.385 inch joint in the longer quoin will be 0.385 × tang. 60° = 0.67 inch longer at the quoin end and 32 inches shorter at the other end, or a resultant of 31.33 inches shorter. This length of 31.33 inches was subtracted from the length of the arris of one stone to obtain the length of the arris at the same joint on the next stone. Had the length of the arris of the bed *farther* from the reference plane been obtained by the general formulæ, the resultant quantities by the short method would have been added instead of subtracted. This shorter method is not precise for either the soffit or extradosal arrises, but the error is inappreciable and no multiplication of errors can take place.

To prevent the multiplication of errors to any great extent two decimal places were retained in all this work. With the granite used final results not greater than ⅛ inch were expected and obtained.

Calculations were made for the three armed keystone at the intersection of the middle of the inverts. The part beyond the center of the main tunnel is equal in width to three bricks and two and one-half brick joints. The length of the arrises next to the other stones were computed by the short method mentioned above and checked by computing their difference in length from the width of the arm between these stones. This difference was subtracted from the length of the longer arris obtained by the shorter method and the remainder was found to be 0.6 inch longer than the length of the shorter arris also obtained by the short method. This was corrected by applying one-half the error to each arris. This error is mostly due to assuming the lengths of two arrises in beginning the work and but little due to errors in the computations. This can be seen if we multiply the half width of the invert, 7.39 feet, by tang. 30° for the shorter quoin and add the length of the assumed arris, 3 feet; from this sum if we subtract the five offsets at the ends of the stones of one foot each we have a remainder of 2.2566 feet. If we multiply 7.39 feet by tang. 60° for the longer quoin and add the length of the assumed arris, 2 feet and 9 inches, and from this sum subtract the five offsets at the ends of the stones of 2 feet and 8 inches each we have a remainder of 2.2165 feet. The difference of these two remainders, 0.04 feet or 0.48 inch, is the difference of length of the middle element in the side tunnel due to not assuming a correct difference in length of the two arrises. Thus it is seen that the original assumption of the length of any arris in the invert fixes the length of all others, except that multiples of four inches may be added to or subtracted from them at will. It is also true that the assumption of the length of one arris fixes the length of all others in the whole intersection.

The following method was used to find the length of the middle elements in the soffit of the stones. In the shorter quoin the difference in values of *b* for the middle element and the soffit element nearest the reference plane was multiplied by tang. 30°, and this product was subtracted from the length of the soffit element nearest the reference plane. In the longer quoin tang. 60° takes the place of tang. 30° above. These middle elements are useful in making the soffit templets, the quoin end of which is a curved line. For the stone at the side wall in the shorter quoin the length of the middle elements is 40.56 inches, and if the quoin end of the template were a straight line it would be 40.58 inches. For the same stone in the longer quoin the true length is 46.71 inches, and a straight line would make it 46.745 inches. The difference on these stones is so small that it may be neglected for the other stones of the invert, as their soffit surfaces are narrower and, at the the same time, the curvature of the quoin is less.

We now have all the elements needed in the invert, but to complete the invert stones we need the lengths of two more arrises at the top of the stone under the side walls. In dividing the side wall for the required number of stones it was found desirable to extend the first stone in each quoin of the invert up the side walls 2.67 inches. Recalling that the reference plane for the side walls is horizontal, the angle between the reference plane and the plane in the top arris of this stone is 21°, 08′, 55″. The angle between the reference plane and a plane in the top element of the invert is 21°, 35′, 17″. The difference of cosines of these angles multiplied by the radius of the side walls = 0.97 inches. In the shorter quoin 0.97 inches multiplied by tang. 30° is the amount this top arris is shorter than the assumed element, and we have 36 inches — 0.56 inches = 35.42 inches for the length of the top arris. In the longer quoin 0.97 inches multiplied by tang. 60° is the amount this top arris is shorter than the assumed element, and we have 33 inches — 1.69 inches = 31.31 inches for the length of the top arris.

To find the difference of length between the assumed arris and the arris in the extradosal surface at the top of the invert stone, the distance of the latter arris from the vertical reference plane should be known. The radius of the extradosal surface of the side walls is 30⅔ feet and the angle between the top of this stone and the horizontal reference plane is 21°, 08′, 55″, and 30⅔ × cos. 21°, 08′, 55″ = 28.60 feet = the horizontal distance of this arris from the center of curvature of the side walls. The horizontal distance between the centers of curvature of side walls and invert is 29.0 — 9.42 = 19.58 feet, and 28.60 — 19.58 = 9.02 feet = the distance of the required arris from the vertical reference plane. The assumed arris is 7.39 feet from the same reference plane, or a difference of 1.63 feet. In the shorter quoin 1.63 feet × tang. 30° is the amount the required arris is shorter than the assumed arris, or 36.0 — 11.29 = 24.71 inches = the length of the required arris. In the longer quoin, using tang. 60° in place of tang. 30°, we see that the amount the required arris is shorter than the assumed arris is greater than the length of the assumed arris, and therefore the stone does not reach through the wall and has no arris on the extradosal surface at the top.

We still need some dimensions for the end templet for the stones under the side walls. It is desirable to make the bottom of these stones level. The joint bed nearest the bottom of the invert makes an angle of 11°, 49′, 42″ with the vertical reference plane, and the difference of radii is 20 inches. Twenty times cos. 11°, 49′, 42″ = 19.58 inches = the vertical ordinate of the templet at this joint. From the difference of the values of b for the inside and outside elements at this bed we have 4.10 inches as the distance this vertical ordinate is away from the arris at the bottom of the stone. The difference of versines of angles at this bed joint and at the middle ordinate multiplied by 29.6 feet is the amount to add to 19.58 inches for the height of stone at the middle element of the soffit, or 19.58 + 1.76 = 21.34 inches. By the same method the height of the stone at the element between the invert and side walls is found to be 23.28 inches. The following method will be found more convenient for finding the other heights. The distance the bottom of the invert is below the top of the side walls is the sum of 10.671 feet from top of side wall to top of invert, 0.936 feet from top to bottom of invert soffit, and 1.667 feet from inside to outside of invert, or 13.274 feet. The bottom of the highest invert stone is 31.267 × versin. 11°, 49′, 42″ = 0.664 feet above this, or the bottom of this stone is 13.274 — 0.664 = 12.61 feet below the top of the side walls. The top arris of this stone is 29 × sin. 21°, 08′, 55″ = 10.463 feet below the top of the side wall, and 12.61 — 10.463 = 2.148 feet = 25.78 inches above the bottom of the stone. For the height of the arris at the extrados we have 30⅔ × sin. 21°, 08′, 55″ = 11.064 feet, and 12.61 — 11.064 = 1.546 feet = 18.55 inches. If we make the extradosal surface of this stone vertical we can find the width and partial widths of the stone for the end templet as follows. As seen above, this surface will be 9.02 feet = 108.24 inches from the vertical reference plane. The bottom element nearer the same plane is 76.91 inches from it, or the stone is 31.33 inches wide on the bottom. Twenty × cos. 21°, 08′, 55″ = 18.65 inches is the width of the part under the bottom side wall stone. As the projection of the side wall portion of the soffit is 0.97 inch, we may call it one inch, and adding we have 19.65 inches. By subtracting 19.65 from 31.33 we have 11.68 inches as the part under the invert.

It was noted above that the stone in the longer quoin does not extend through the side walls, and the end templet to fit the portion under the bottom stone of the side wall will be smaller than for the same stone in the shorter quoin. The length of the top arris is 31.31 inches and this multiplied by tang. 30° = 18.07 inches is the width of the bottom under the bottom side wall stone. Twenty × $\frac{18.07}{18.65}$ = 19.38 inches is the width of the top bed. 25.78 — 18.55 = 7.23 inches, and $7.23\left(1 - \frac{18.07}{18.65}\right) = 0.22$ which if added to 18.55 will equal the height of the outside arris, or 18.77 inches.

Now all the dimensions necessary to plot the stones of the invert and make their templets are known, but it is desirable to be able to test the finished stones and to do this the angle formed by the intersection of the two planes of the coursing joints of each stone should be known. If we suppose a plane passed through the soffit arrises meeting at a quoin it will be horizontal and will form a spherical triangle with the two adjacent bed planes. In this spherical triangle the side opposite the required angle is known, being 120° in the shorter quoin and 60° in the longer one. The two angles adjacent to this known side are equal to each other and to 90° minus the angular distance of the beds from the reference planes. For the invert the average angular distance of the two stones at a joint will be exact enough for both stones. We have

$$\text{Cos. } B = -1 + 2 \sin.^2 A \cos.^2 60^\circ$$

for the shorter quoin, and

$$\text{Cos } B = -1 + 2 \sin.^2 A \cos.^2 30^\circ$$

for the longer quoin. If this value be negative it is the cosine of the supplement of the required angle, if positive it is the cosine of the required angle. These angles are given on the plate for the long quoin only.

Since in the above the writer has described in detail the work on the invert, it will be desirable to notice the differences only in the other parts.

SIDE WALLS.

In the side walls 6 brick courses will give 16.056 inches for the width of the soffit of a stone and a stone joint. Eight times this will leave 2.67 inches to be cut on the top of the invert stone. As the soffit width of a stone in the side walls is 15⅝ inches, we have 16.056 — 15⅝ = 0.431 inches for a stone joint.

For the side walls the reference plane is a horizontal plane through the axes of the tunnels and at the top of the side walls. The angular distances, α, from the reference plane to the beds of the stones and the middle element on the soffit of each stone were calculated. The horizontal distances of the arrises of the top beds from the center of curvature of the side walls were calculated by the formula $R \times \cos \alpha = b$. This is the same formula that was used for obtaining the value of b as in the invert, but, the reference plane being 90° in a vertical arc from the reference plane of the invert, cos. α takes the place of sin. α. If 19.58 feet be subtracted from these values of b we get the horizontal distances from the axes of the tunnels to the top bed.

The difference of length at the quoin end of two soffit elements in the side walls is found by the general formula given above, recalling that cos. α is used instead of sin. α in the value of b. For the shorter quoin diff. of cosines × R tang. 30° is used, R being the radius of the soffit elements. For the longer quoin tang. 60° takes the place of tang. 30°. By this formula the difference of length at the quoin end between each soffit arris and the assumed element at the top of the invert is found, and also the difference of length at the quoin end of the middle elements and the assumed element. The necessary difference in length required at the end of the arris in order to make a good brick bond was decided upon and combined with the above results to get the length of each soffit arris. That the stones on the shorter quoin might not be too heavy their length in the main tunnel were made 8 inches shorter than in the side tunnel.

For the difference in length at the quoin end of a soffit arris and the corresponding extradosal arris the formula, 20 inches × cos α tang. 30°, is used for the shorter quoin, and the same with tang. 60° instead of tang. 30° for the longer quoin. In all cases this is to be subtracted from the corresponding soffit arris, and a negative result shows that the stone is too short to reach the extradosal surface. This is the case with two stones on the longer quoin. As these stones were over two and one-half feet long, it was not desirable to increase their length 12 and 16 inches in order to retain the required bond, that they might reach farther through the wall. The space between the side walls and the solid rock at the intersection as elsewhere was filled with rubble masonry laid in cement mortar.

The dimensions of the end templet for the stones in the side walls are obtained by making the width on the soffit $15\frac{5}{8}$ inches, the radius of the soffit being 29 feet. The radius of the extradosal surface being 20 inches greater, the width on this surface is proportionally greater; it is 16.53 inches. For the two stones on the longer quoin that do not reach through the wall, the arris formed by the intersection of the two end planes is a vertical line. If we suppose a horizontal plane passes through the top soffit arris of one of these stones the line of its intersection with the plane of the quoin, and the line of its intersection with the end plane of the stone will form a right-angled triangle with the line of the arris, having an angle of 30° at its apex, or quoin end. If we represent its base by k, then k will equal the length of the soffit arris × tang. 30°. The angle formed by k with the arris between the bed and the end plane of the stone is the same as the angle between the bed plane and the reference plane; this angle is α. If we represent the length of this arris by m we have $m = \frac{k}{\cos \alpha}$. If we represent the vertical distance between the extradosal ends of k and m by n, we have $n = k$ tang. α. To find the value of m and n for the top and bottom arrises of the two stones that do not extend through the wall, we have only to use the value of k derived from the length of the proper soffit arris and the corresponding value of α. In making the end templet for these stones it will be recalled that m is perpendicular to the soffit surface and the extradosal ends of the two m's on the same stone are joined by a straight line which is vertical when the stone is in its position in the wall. As a check, the length of this line is equal to 29 feet × diff. of sin α for bottom and top beds of the stone plus the diff. of n's for bottom and top soffit arrises.

The angles desired for testing the cutting of the beds of the stones were obtained by substituting in the same formula as for the invert. In this case the two angles adjacent to the known side are equal to each other and to the angular distance of the beds from the reference plane. The difference of the angles between two stones at any joint being so small, the values cos B and the angle B were found for the middle of the joint and used for both stones at this joint.

CROWN ARCH.

The width of the tunnels at the top of the side walls is 18.84 feet and the crown arch is full-centered. This will require 131 brick courses of 2.7075 inches each to form the soffit surface. By dividing this by 6 we find that 21 stones of 6 brick courses each and 1 of 5 brick courses will be required to form the sides of the quoins in the side tunnel. These 22 stones also form the side of the quoins in the main tunnel, but as each stone in one quoin has a stone opposite it in the other quoin they reach only 66 brick courses of the soffit. This is one-half a brick course beyond the intersection of the quoins at the top of the soffit. This arrangement makes a joint come at one-half a brick course from the top and divides the keystone in two parts. To have made a keystone of one piece to be placed opposite the top eleven courses of brick in the side tunnel and to extend in both directions in the main tunnel to make the necessary brick bond, would have made it too heavy to handle in the confined space at the top of the tunnels. It was, therefore, made of two pieces, one of which is opposite six brick courses in the side tunnel and six brick courses in the main tunnel in the shorter quoin, the other is opposite five brick courses in the side tunnel and six brick courses in the main tunnel in the longer quoin. A plain stone voussoir equal in width to six brick courses was put in the soffit of the main tunnel opposite the joint between the two pieces of keystone. Thus it is seen that the stone in this intersection extends from 3½ brick courses beyond the center of the main tunnel on the invert soffit to 6½ brick courses beyond the center of the main tunnel on the crown arch soffit and forms the 6 quoins between these points.

For convenience a vertical plane through the axis of the tunnel was taken for the reference plane for the crown arch, the same as for the invert. The arc of the soffit covered by a stone and a stone joint is 16.245 inches. The soffit of the stone is 15⅞ inches wide, leaving 0.37 inches for the stone joint in the crown arch. With the length of the arc of a stone and a stone joint the angle between the reference plane and the top bed of each stone was obtained. With these angles the values of b were obtained by the same formula as for the invert. It will be recalled that b is the horizontal distance from the reference plane to the top arris of each stone.

In this part of the work more than in the others it was desirous that no stone should be heavier than necessary. For this reason the length of the keystones were found first. The length of the top element of the arch between a vertical plane in the longer quoin and a vertical plane perpendicular to the side tunnel at the intersection with the quoin plane of the lowest extradosal arris of the keystone in the longer quoin, was calculated. To this length was added the lap of the keystone of the longer quoin upon the stone next below it. From this was subtracted 16 inches as the distance from the intersection of the top elements of the tunnels to the end in the side tunnel of the keystone in the shorter quoin. To this last length multiples of 4 inches were added to give the distance on the top element of the ends of the successive stones in both quoins. By assuming the lap of keystone of the longer quoin upon the stone below it of a certain length, the exact requirements of brick bond at the top of the side walls on one side could be obtained, the other side would still have the same difference as shown in the invert. It was so much more important that the top stones should be as light as safety would permit, that the only attention given to the brick bond between the bottom stones of the crown arch and the top of the side walls was that it should not be less than 8 inches. Throughout all the design much attention was given to make the stones as light as possible, but security was not sacrificed for it. The smaller piece of the keystone contains 16½ cubic feet, the larger 19 feet, and excepting these, no stone in the quoins of the crown arch is as small as the larger piece of the keystone. The average size of the stones in this intersection is very nearly one cubic yard.

The difference of the lengths of two arrises at their quoin ends was obtained by the formula, diff. of length of arrises = diff. of sines × R tang. 60° for the longer quoin, and tang. 30° was used instead of tang. 60° for the shorter quoin, the proper radius being used for the soffit and extradosal surfaces. By the same formula the length of the middle elements of the soffit of each stone was found. The radius is so small that the middle element was needed in constructing the templets.

The two parts of the keystone extend beyond the top of the main tunnel one and one-fifth inches. The lengths of the eight arrises near the top of the tunnels were calculated by the same methods as the other arrises, the arc being comparatively small.

The angle between the beds on the same side of a stone was calculated by the same formula as used before for this purpose. The radius of the soffit being so small it was necessary to calculate these angles for both top and bottom side of each stone.

The stones were cut and marked in the granite quarries near Wilmington, Del., and shipped to Niagara Falls. Each stone was marked with a letter and two numbers. The letter indicated that it formed part of this intersection. The first number denoted the quoin and the second, the position in the quoin.

The making, above ground, of the parts of the center for the crown arch was somewhat difficult. Only one supporting post away from the side walls could be used and the lower chords of the quoin ribs rested on it with the lower chord of a rib from the opposite side of the main tunnel. The inner ends of the lower chords of the other ribs rested on the lower chords of the quoin ribs, and were inclined chords if the outer ends reached the side walls. The web members and upper chords were made of several thicknesses of ⅞ inch pine nailed together. Each piece in the quoin rib forming the upper chords, was cut with a changing bevel to its proper elliptical curve. After nailing together they formed an angle of 120° at the bottom of the shorter quoin, and one of 60° at the bottom of the longer quoin. These dihedral angles gradually increased to 180° at the tops of the ribs. While above ground, intersecting points were marked on the quoin ribs and the proper bevels were cut on the ends of the upper chords of the other ribs. When needed the parts were sent into the tunnel and erected without obstructing the other work in the tunnel.

PNEUMATIC TRANSMISSION IN UNDERGROUND TUBES.

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While a description of present practice in New York City and of some of the problems encountered in the operation of the large mail tubes there, during the writer's superintendency, is the chief purpose of this paper, it may be of interest to first mention briefly what was accomplished in the general field of pneumatic transmission prior to the establishment in 1897, of the various eight-inch tube systems under the Batcheller patents.

EUROPEAN SMALL TUBES.

First let it be said that, as regards large tubes—six and eight inch—and the transportation of mail therein between the post office stations of a city, American practice is the only practice. The European tubes are nowhere larger than three-inch and are used exclusively for the handling of telegrams and messages in connection with governmental control of the telegraph. In London, Paris, Berlin, Vienna and some of the smaller cities, telegrams are thus transmitted from one district to another more cheaply and quickly than by wire ; and, in addition, a large personal message system within the city has been developed.

Early History. The first pneumatic transmission of which we know occurred in 1667, and was in the nature of an experiment made by Denis Papin and described by him in a paper presented to the Royal Society of London. Further experiments by widely separated investigators followed at long intervals, and it was not until 1853 that the first successful commercial application was achieved. In that year an English engineer, Latimer Clark, designed and constructed for the Electric and International Telegraph Company of London a line of single tube 1½ inches in diameter and a few hundred yards long. Five years later the same company built a system of two 2¼-inch tubes three-quarters of a mile in length for transmission in both directions ; and, for the first time, compressed air was used for propulsion in one direction, a vacuum being retained for the other.

London System. This 1858 line was the nucleus of the present 34 miles comprised in the London system. The tubes are from 2¼ to 3 inches in diameter and are of lead encased in cast iron pipes for protection. The joints of the lead tubes are made smooth and uniform by soldering over a heated mandrel. The carriers consist of light felt with leather flare-pieces, front and rear, fitting the tube.

Berlin, Paris and Vienna. The Berlin tubes are somewhat smaller than those in London. In both cities the radial lay-out is used, the tubes branching in pairs from a central station. The systems of Paris and Vienna are planned on the polygonal-circuit system,—the carriers moving, in one direction only, around the sides of a polygon, and passing in turn through each station.

United States Small Tubes. In this country the Western Union Telegraph Company, since 1876, has used small tubes in New York City for the transmission of telegrams between its main office and some sub-stations. Small tubes are also common in the cash systems of our department stores and, to an increasing extent, in large office and public buildings, and it is not necessary here to discuss their essential features.

THE PHILADELPHIA SIX-INCH LINE.

Change to Increased Size Difficult. The large mail tubes of the United States present greatly increased difficulties over their smaller and therefore simpler European prototypes. As an instance, the carrier in the Paris system weighs, when filled to its capacity of 35 messages, a trifle over two-thirds of a pound. A carrier of New York's eight-inch lines weighs nearly 16 pounds empty and between 25 and 30 pounds full,—its capacity being from 400 to 600 letters. This difference is graphically shown in *Fig. 1*, which is a photograph of the different carriers. It may readily be conceived, therefore, that the jump to a six-inch system was no easy one. It was accomplished successfully however, in the winter of 1892–93, by B. C. Batcheller, an American mechanical engineer, who was chosen as engineer in charge of construction by a Philadelphia company about to lay a six-inch line between the general post office of that city and a sub-station in the Bourse.

Principal Problems. When Mr. Batcheller came to build the system he found radical defects in the design of tubes and apparatus, and much of it had to be re-designed by him. The more important problems requiring solution were the need for a tube with absolutely smooth interior, a carrier light though strong enough to stand the hard use it would receive, and the starting and stopping of the carriers. In the last, the European method of interposing some solid obstruction was, of course, inadmissible, and the great momentum of the much heavier

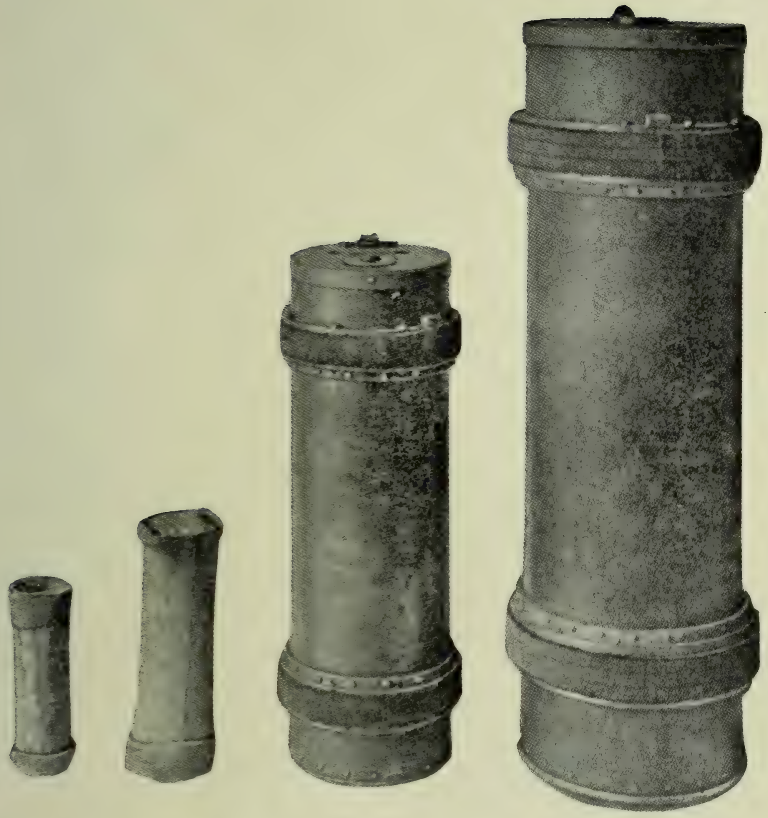


FIG. 1.

1.—Carrier used in the Berlin system.
2.—Largest carrier used in the London system.
3.—Six-inch carrier used in the first Philadelphia system.
4.—Eight-inch carrier used in New York, Boston and Philadelphia.

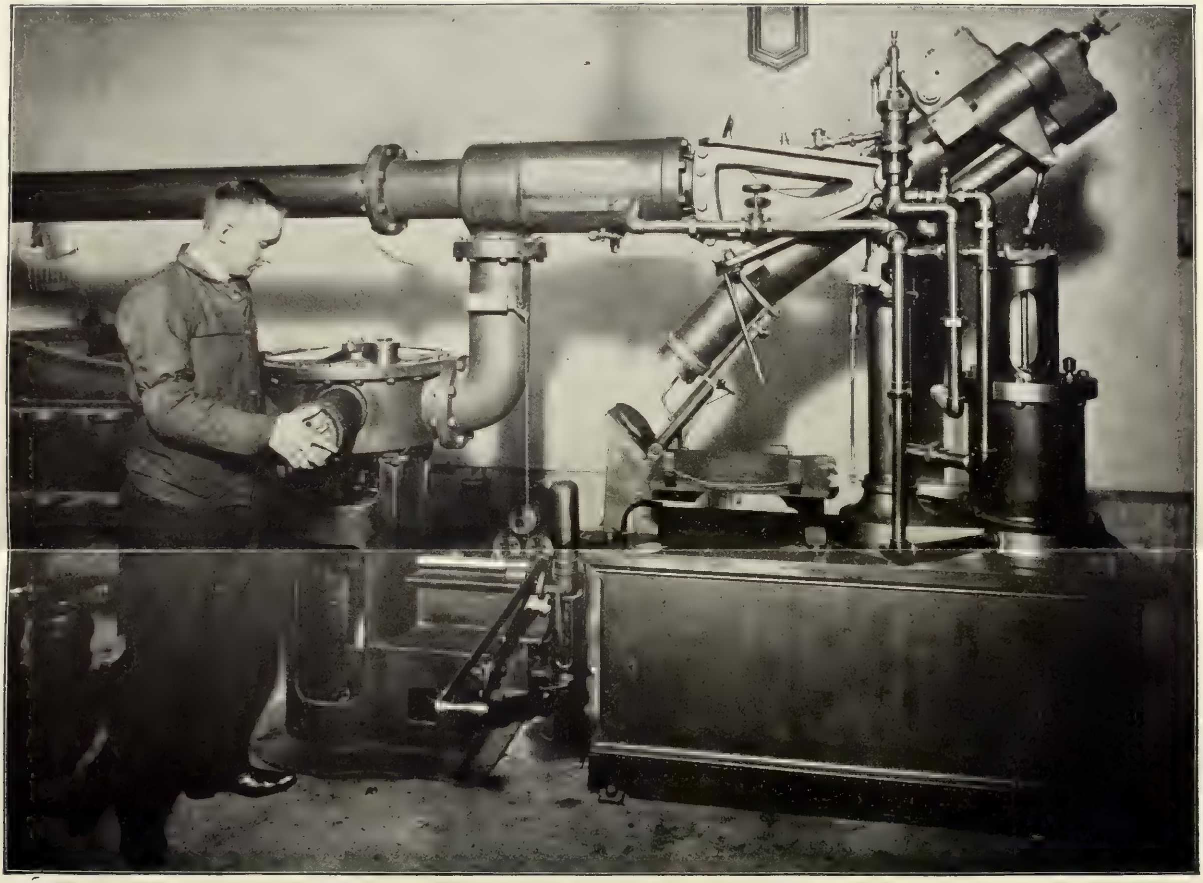


FIG. 2. SIX-INCH TUBE. SENDING AND RECEIVING APPARATUS, PHILADELPHIA, PA.

and more rapidly moving carrier made necessary the use of the present automatic receiving apparatus, whereby the carrier is brought gradually to rest by means of an air cushion.

Description 6-inch Line. This six-inch line is the basis of all the eight-inch systems afterward constructed. Its features are, in the main, identical and will be described later on under those of the larger tubes. The transmitting apparatus only is different in form though not in principle. It consists practically of a large valve turned in a circular casing by hand, so that a carrier may be inserted off the line of the tube and brought into it and discharged without interfering with the continuous flow of the air current. The line is what is known as a loop or one-compressor circuit with closed receiver at the terminal opposite from the compressor. This latter is of the Clayton type, 25 H.P., and supplies 800 cu. ft. of free air per minute, at 7 lbs. pressure.

While the eight-inch lines are essentially similar to the original six-inch one, they are of much greater capacity and importance, and the heavier carriers, increased momentum, and the larger and more complex apparatus present operating problems in no way comparable with those of the Philadelphia small system.

EIGHT-INCH LINES.

In Philadelphia, Boston and New York. In 1897 Congress authorized the construction of systems of eight-inch tubes in Philadelphia, New York and Boston for mail carrying purposes. The Philadelphia circuit is a one-compressor line connecting the General Post Office with the Pennsylvania Railroad station, taking in on the way the Reading Terminal as an intermediate. The Boston line, between the General Post Office and the Union Railway station, is also a short one, having one compressor, and, at the opposite terminal, a closed receiver.

In New York city three circuits were constructed, all centering at the General Post Office,—one running to the Brooklyn Post Office, one to the branch post office station in the Produce Exchange building near the Battery, and one to Branch " H " on Depew Place just across the street from the Grand Central Station. All the above lines were constructed under the Batcheller patents, and all are plenum systems.

THE NEW YORK CIRCUITS.

Line to Produce Exchange. This was the first eight-inch circuit built, being completed early in October, 1897, two months after ground was broken.

The one compressor is at the G. P. O. end, where the terminal machinery consists of an open receiver and a transmitter. The Produce Exchange station is equipped with a transmitter and closed receiver. The air-current from the compressor flows out one tube to the Produce Exchange, through the closed receiver there, and back in the other tube to the G. P. O.

The length between terminals is 0.775 miles, and the average time of transit one way is 1 min. 33 sec. at the speed used on all these circuits, 30 miles per hour, or 44 ft. per sec. The initial pressure is 6 lbs. at the G. P. O., the pressure falling to 3 lbs. at the Produce Exchange.

Line to the Grand Central Station. As first constructed, this circuit was continuous between the G. P. O. and Branch "H", with branches to three intermediate stations, P. O. Branch "D", 8th Street and 3rd Avenue; "Madison Square", 23rd Street and Madison Avenue; and "F", 28th Street and 3rd Avenue. These branches could be thrown in or out of the trunk line by switches operated from the intermediate stations. When the circuit was in complete operation, however, it was designed to have the carriers pass through each station in turn, their separation being accomplished by the intermediate machines themselves in a manner to be described later on.

The original circuit was 3.64 miles long between terminals, by way of the trunk, and 4.41 miles by the route in and out each of the intermediate stations. With the smaller distance it took 7 min. 16 sec. for a carrier to pass from G. P. O. to Branch "H", and 8 min. 49 sec. by way of the intermediates. The initial pressure was 11½ lbs., with a compressor of 90 H.P. at each end of the line. A transmitter and open receiver constituted the terminal station equipment, and there were two intermediate machines at each branch station.

The circuit has since been divided at the Madison Square station, so that it now consists of two lines, each terminating at that point. The reasons for the change will be discussed in another part of this paper. It is interesting to note that the original circuit was the longest line, either of small or large tubes, in existence.

Line to Brooklyn. This connects the New York and Brooklyn General Post Offices by way of the Brooklyn Bridge. At each end of the line are a compressor, open receiver and transmitter. There are no intermediate stations, and the circuit is a good example of the simplest pneumatic tube construction for large mail lines. The length one way is 1.65 miles, of which one mile is on the bridge. Each compressor is of about 45 H.P., and the initial pressure used is 5½ lbs., which gives as the time between terminals, 3 min. 18 sec.

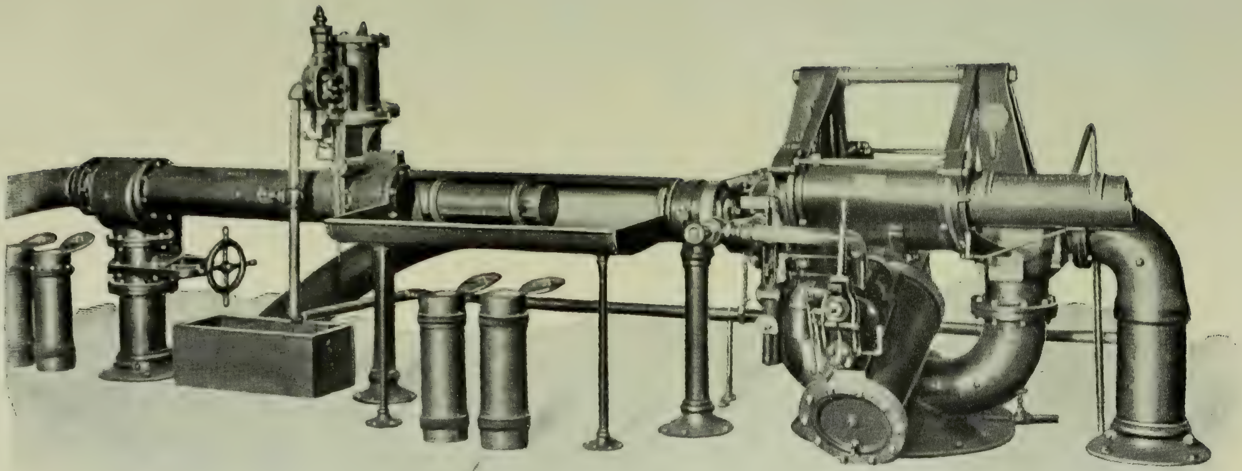


FIG. 3 SENDING APPARATUS AND OPEN RECEIVER. PRODUCE EXCHANGE LINE. MAIN POST OFFICE. NEW YORK CITY.

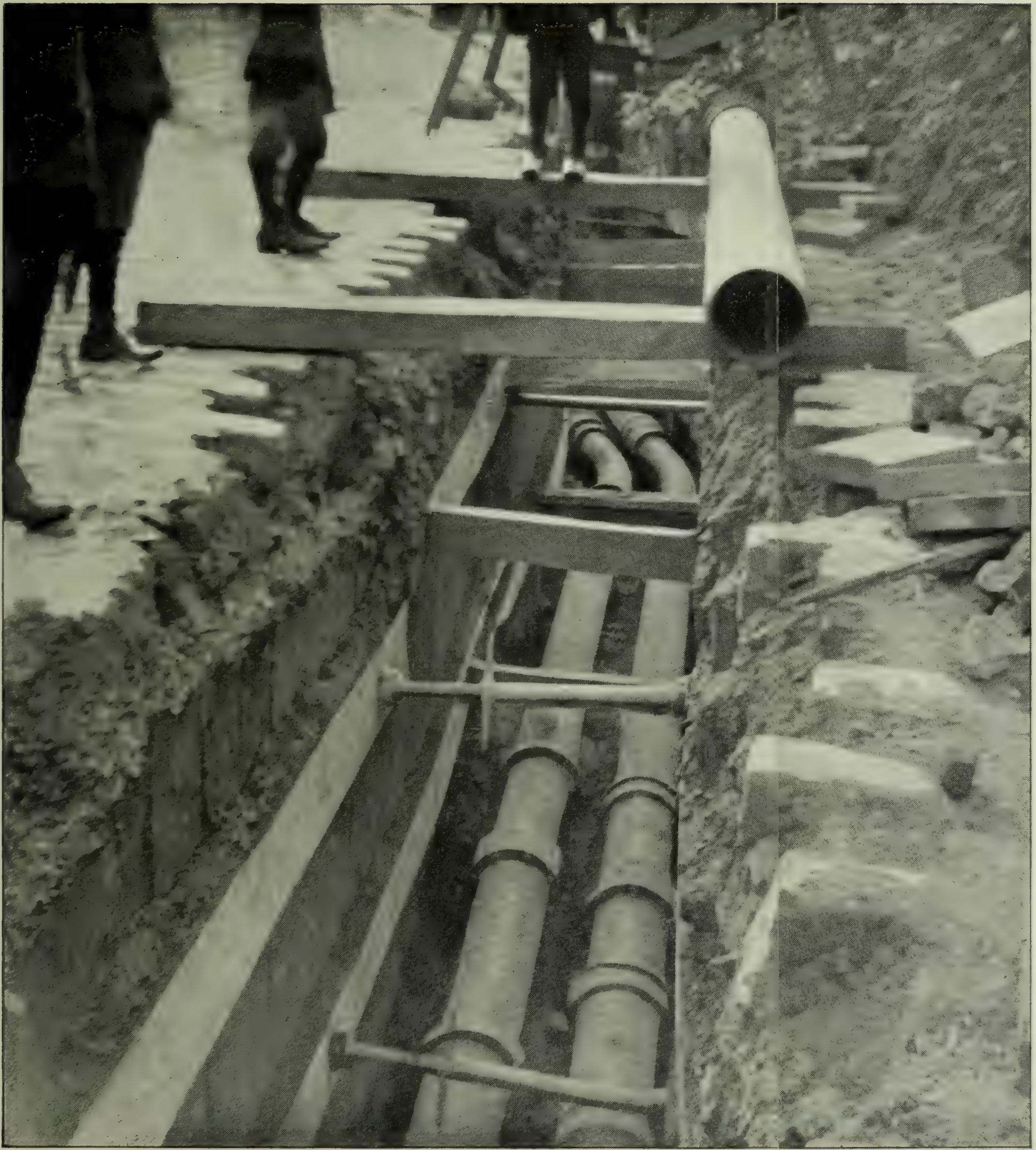


FIG. 4. LAYING THE TUBES.

TUBES AND LAYING.

Material and Dimensions. The tubes used in all the eight-inch systems are specially cast water pipe bored carefully to a smooth interior surface. This pipe is $\frac{9}{16}$ in. thick before boring, and weighs 60 lbs. per ft., or 720 lbs. per 12 ft. length. It is cast 8 inches inside measurement and bored to $8\frac{1}{8}$ inches. The boring is done very accurately in Philadelphia by an ingenious process devised by Mr. Batcheller early in the construction work on the six-inch line.

Wrought iron tubes were first supplied for the line, but they had to be discarded on account of their unevenness and many imperfections. At the time, the bored cast iron pipe was deemed a temporary expedient, but nothing better has since been found. Wrought iron is too rough, brass too expensive, and steel tubing has not yet been drawn uniform enough. The disadvantages of the bored cast iron are expense and rusting.

Accuracy in Boring. Smoothness and uniformity of interior are essential because of the rigidity of the carrier and to minimize friction. The Philadelphia factory bores to a variation in diameter of 0.01 inch, measured by passing a hardened steel mandrel through the finished tube. In laying, another mandrel is used which is $\frac{1}{64}$ inch smaller than the tube. To insure a straight and continuous interior the spigot end of each tube is machined to fit a counter-bore turned in the bell of the next length.

Bends. For turning street corners and passing obstructions bends of seamless brass tubing, $8\frac{3}{8}$ inches in diameter and $\frac{1}{8}$ inch thick, are used. All bends are of the one radius, 8 feet. In order to preserve a uniform cross-section when bending the tube, it is filled with hot resin and bent between rolls after the resin has cooled. Any slight inequalities are removed with a mandrel. After the ends are cut and fitted with brass flanges for joining, the bend is complete.

The tubes are laid side by side in the trench with no other support than wooden blocking. The fragile bends only are bricked or concreted in for protection. In New York the maze of underground pipes in some of the downtown streets left little room for the pneumatic lines ; but, in addition to the bends and their combinations, the tubes themselves can be deflected at each joint as much as 48 minutes, or 2 inches in 12 feet, making the avoidance of obstructions ordinarily easy.

Jointing. Extreme care is observed in jointing and in keeping the interior of the tubes clean and free from rust. The joints are lead, and, in caulking, the laying mandrel is kept in the tube to be pulled through afterward for testing the joint, the interior of the tube, and the deflection. Before jointing, each spigot is rammed tightly home into its opposite counter bore. In spite of the closest inspection, the trial of the first carrier through the completed tube is always a matter of considerable concern to the engineers charged with the work.

Caulked joints have the advantage over flanged ones of providing a slight though sufficient play, and, in the case of tubes underground, of taking care of the expansion and contraction without the need of special devices.

Switch Vaults, Gage Pipes and Drips. The physical equipment of the line proper is completed with vaults to contain the switches where branches turn out from trunks ; gage pipes (small pipes every 300 ft., tapped into the tube and run into boxes at the street surface) for testing pressure and locating obstructions ; and, at all low points, drips for the collection of moisture of condensation. These are boxes holding about five gallons of water, and blown off, through small pipes leading to the street, by the tube pressure. In New York the drips are attended to at stated intervals by a lineman, who is also charged with a vigilant inspection of all street openings near the tube.

DISPATCHING AND RECEIVING APPARATUS.

Transmitter. The transmitter for the Philadelphia six-inch line has already been described. That for the eight-inch system consists of a swinging cradle holding two short sections of tube (barely longer than the carrier), at opposite ends of an arc of about 45 degrees. The transmitter in the position for sending is shown in section in Fig. 5.

A continuous air current flows from the compressor to the underground line, passing through B, one of the short sections, on its way. When a carrier is to be dispatched, it is inserted in the opposite short piece C, being supported and guided in the operation by a metal trough. An operator then pulls the lever K, and the cradle swings over between the fixed guides L until the section C takes the place of B in the line of the air current, thus discharging the carrier. During the motion of the cradle back and forth, the air current is shut off by the plates M (which form part of the frame connecting the short pieces B and C) and deflected through a by-pass D.

The power to move the cradle is furnished from the air pressure of the tubes. This pressure is the motive power in the case of all the terminal apparatus of the system. The hand lever K operates a controlling valve which admits air to the cylinder and piston H to swing

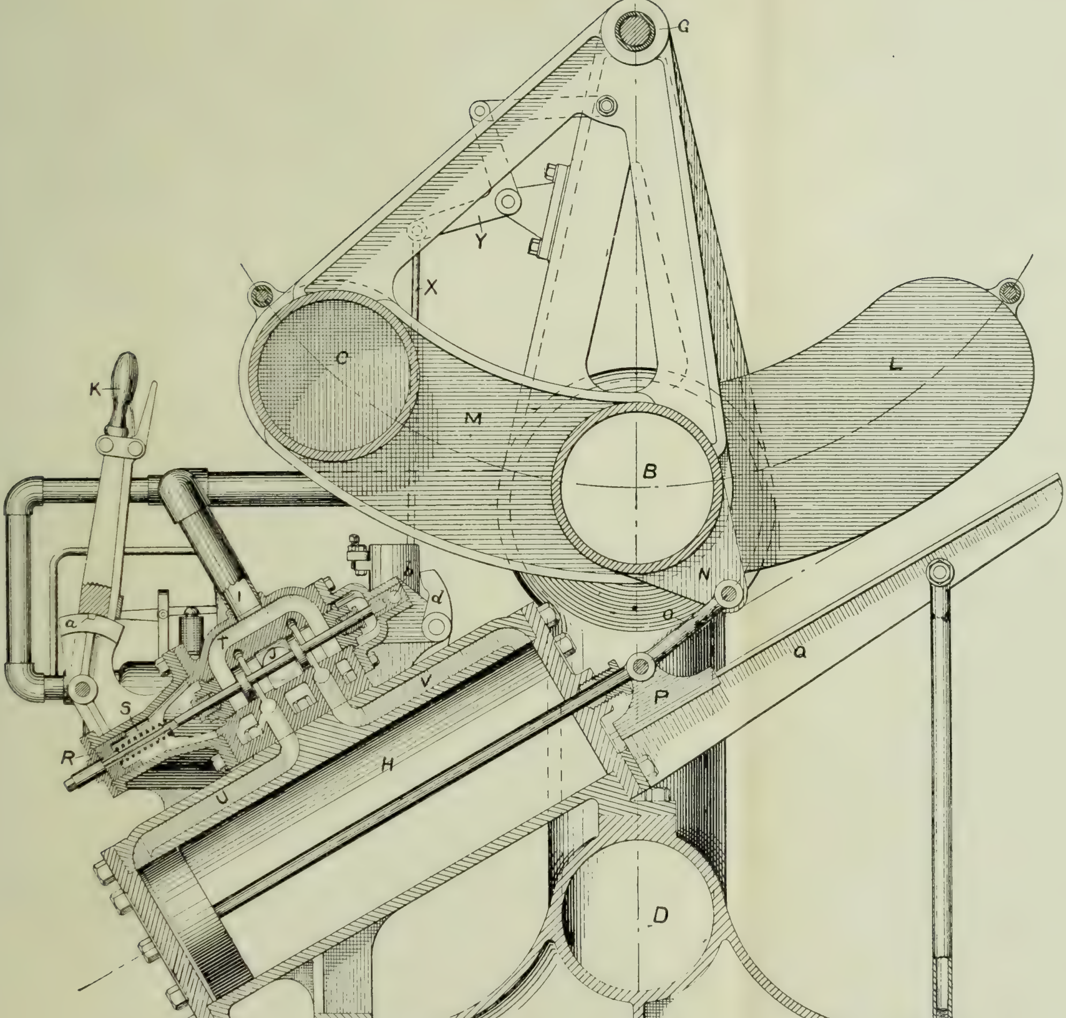


FIG. 5. CROSS-SECTION OF SENDING APPARATUS.

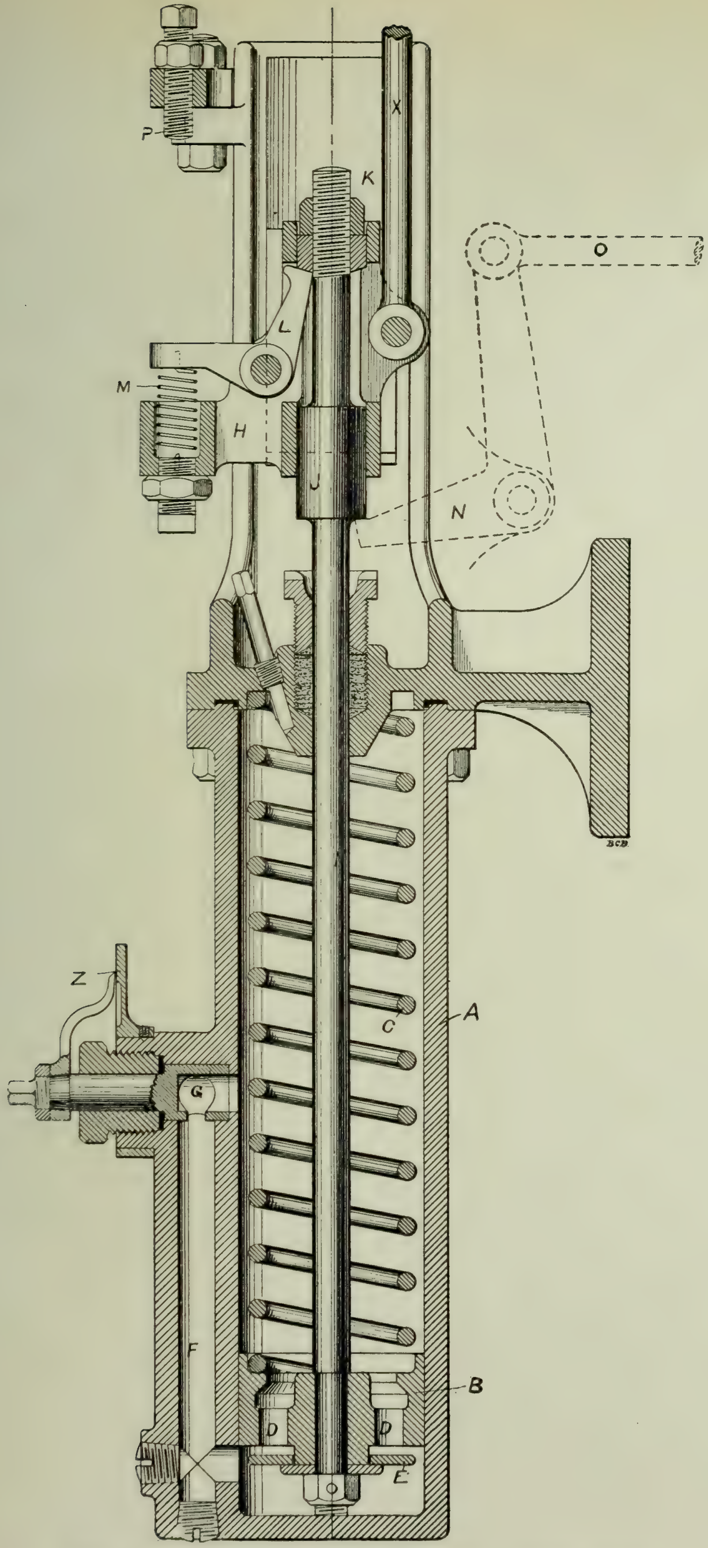


FIG. 6. TIME-LOCK FOR SENDING APPARATUS.

the cradle. As the carrier leaves the transmitter, it trips a steel finger, which throws back the lever and allows the cradle to return to its former position.

Time Lock. A regular interval must be preserved between the dispatch of carriers, made great enough to prevent as far as possible the overtaking of one carrier by another and their appearance at the receiving station so close together that an accident may result. For this purpose a time lock is attached to each transmitter consisting (see Fig. 6) of a piston in an oil dash pot moved upward against a spring when the carrier is dispatched. The upward stroke of the piston locks the controlling valve of the transmitter and prevents the sending of any more carriers until the piston returns to place, which it does by the spring pulling it slowly downward against the oil in the cylinder. As the piston descends, the oil escapes through a small hole, the opening of which can be adjusted to give any desired speed to the piston and a corresponding time interval between dispatching. If the operator pulls the lever while the transmitter is locked, the cradle will not move until the set time interval has expired, when it will go over automatically and discharge the carrier. So far as dispatching is concerned, the operator's only work is pulling the lever, after which he may attend to other duties until the transmitter acts.

Open Receiver. At the other end of the tube from the compressor the pressure has fallen to nearly atmospheric, and the carriers are stopped and taken out of the tube by means of an open receiver. It consists of an air cushion and dead end formed by closing the end of the tube by a gate and taking the air current through a by-pass. As the carrier comes into the dead end, the air current behind it is deflected downward through the by-pass into the engine room, the degree of such deflection being regulated by a damper controlled from a hand lever under the dead end, and determined by the small amount of air required behind the carrier to force it out when the gate has opened. As the air cushion in the dead end overcomes the momentum of the carrier and is compressed ahead of it, the increase in pressure operates a small valve that admits air to the under side of a piston in a cylinder directly above the gate. The gate is attached to the piston and opens as the latter rises, allowing the carrier, under the influence of the few ounces of pressure behind it, to be discharged on to the receiving pan. The carrier as it passes under the gate lifts a steel finger hanging downward in the tube, which, in dropping, admits air to the upper side of the piston and thus closes the gate.

In the event of carriers overhauling one another in the tube, the finger has no chance to fall so long as the carriers come in together, and the gate remains open until the last one is out of the way ; but with the carriers a short distance apart, the finger has had an opportunity to release the gate which may be falling just as the following carrier comes under it. A carrier is not often crushed in this manner, as the open receiver occupies less than one second for its complete cycle, and one carrier is usually out of the way before the next arrives.

Carriers are sometimes received with greatly varying force depending upon the length of line, its conditions as to lubrication, its degree of loading, the size of bearing rings of the carriers, the weight of mail they contain, and the cleanliness of the interior of the dead end. In extreme cases one carrier may stick under the gate, requiring the closing of the by-pass damper to blow it out, and the next may be discharged into the pan with great force, although the damper has been reset in the meantime. To prevent injury to the carriers a buffer is provided in the receiving pan, consisting of a spring and small air cushion or of a stationary leather bag stuffed with cotton waste.

Closed Receiver. In a loop or one-compressor circuit, as described in a previous paragraph, the pressure at the terminal away from the compressor is too much above that of the atmosphere to admit of the use of the open receiver. The carrier must be removed from the tube without interrupting the continuous flow of the air current or allowing it to escape into the station with the noise and rush of air that would ensue. At such stations the closed receiver shown in Fig. 7 is used. It is practically a closed section of tube, long enough to receive two carriers, and hung on trunnions so that it can tilt to an angle of 45° from the horizontal. The carrier runs directly into this short section or barrel and is brought to rest by the air compressed ahead of it,—a relief valve in the closed end of the receiver preventing the carrier from rebounding.

The added pressure ahead of the carrier operates a small piston slide-valve which admits air into a vertical cylinder below the closed end of the barrel. The piston of this cylinder is connected to the end of the receiver so that as the piston is forced upward by the air under it, the barrel is tilted and the carrier allowed to slide down through the open end on to a steel pan pivoted at an angle of 45°, and depressed to the horizontal by the weight of the carrier, which then rolls off on to the receiving table. As the barrel tilts, a curved plate carried by it above its open end closes off the end of the tube, and the air current flows on down a by-pass and through the transmitter of the return line.

The controlling valve is reversed by the downward movement of the

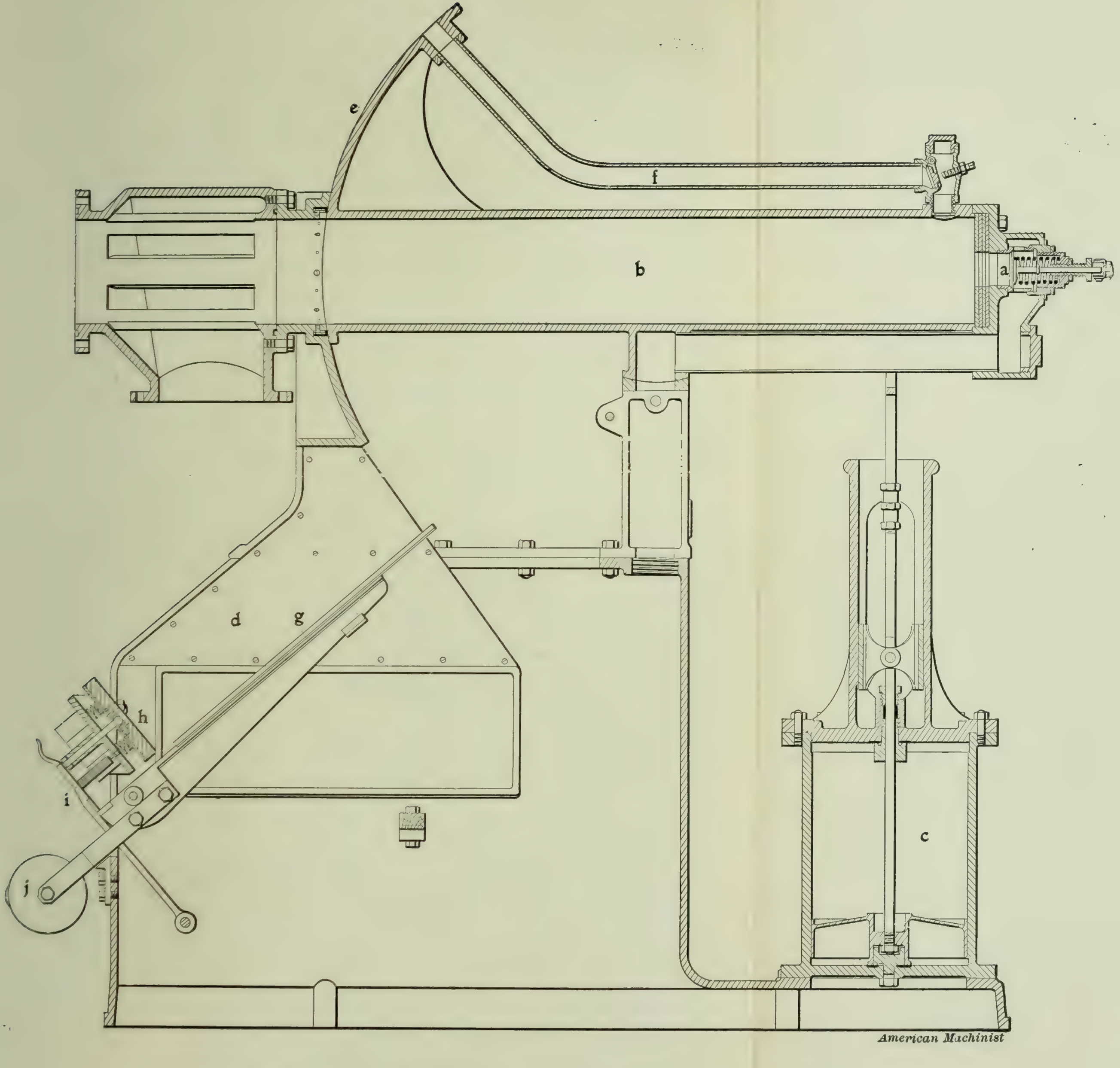


FIG. 7. LONGITUDINAL SECTION OF CLOSED RECEIVER.

pivoted pan under the weight of the carrier and the barrel returns to the horizontal again, the complete operation requiring from 3½ to 4 seconds. If a carrier comes into the station before this time, it may so wedge the receiver as to cut off the air current and thus cause a blockade.

Intermediate Machines. For stations intermediate between two terminals an apparatus is used more complicated than any yet described. In effect it is a combination of closed receiver and transmitter with the addition of an electric attachment for separating the carriers by stations. The dead end in which the carrier is brought to rest is contained in a large wheel that revolves to two positions, one at 45° when the carrier is to be thrown out at the station, and the other at 90° when it is to be sent on through the tube. This wheel is operated by a cylinder and piston with pressure from the line; and a transmitting cradle set vertically, instead of horizontally as in the ordinary dispatcher, is moved in a similar manner. When the wheel is revolving, the air current is deflected through a by-pass as in the closed receiver.

Separating the Carriers. As a carrier reaches the station, it runs up a long reverse bend from the street, and is brought to rest in the dead end of the wheel. A relief valve allows the carrier to come up gently against the head of the dead end where it touches two contact points set in a case in the head and held in place against springs. If the carrier is not meant for the station it will revolve the wheel through 90° until the carrier comes over a vertical outlet and is discharged into the tube. If destined for the station, a metal disc attached to the bottom of the carrier will close an electric circuit between the two contact points, causing the wheel to stop at 45°, where the carrier is ejected. This is a bare outline of a rather involved operation which, in the case of the 90° swing, requires from 5 to 7 seconds for its completion.

On the long Grand Central circuit with the three intermediate stations as originally contemplated, the distance between the contact needles was varied to correspond with a different size disc for each station, the largest for Branch "D", the medium size for Branch "Madison Square" and the smallest for Branch "F". A carrier without disc would pass in turn through the wheel at each station until it reached the terminal; with disc it would be thrown out at the station whose contact needles were spaced to correspond or were closer together than the size of the disc. In the latter instance it was necessary to redespatch the carrier.

Preserving Carrier Intervals. Besides the separation process, the intermediate machine has another function,—that of maintaining the intervals between carriers. A time lock on the same principle as the transmitting one previously described, acts to automatically throw in the 45° stop when a carrier, with or without contact disc, arrives at the station too near to the one ahead. The ejected carrier will then have to be retransmitted by the operator after the regular time interval has been obtained.

Switches. The switches for the intermediate stations on the Grand Central circuit are shown in Fig. 8. They are operated by air power,—a valve being opened and tube pressure admitted to the cylinders by the throwing of an electric switch at the station. It must be borne in mind that the switches in the Batcheller system are not used to separate the carriers, but only to cut a branch station in or out from the trunk as need or convenience requires.

THE CARRIER.

It is safe to say that the carrier is the most troublesome problem of a system of large tubes. On the New York lines it is still the weakest point of the equipment in spite of the time and thought given to its development both by the inventor and the operating engineers, and of the many changes made in the original form.

Carrier Described. Its main features are a steel cylinder $\frac{1}{32}$ inch thick, 24 inches long and 7 inches in diameter, having at one end (the forward one as the carrier goes through the tube), a convex bottom holding a buffer, the other end being closed by a hinged lid. On the outside of the shell are two double bands of metal holding a ring of some packing material. These are called packing or bearing rings, and comprise its frictional surface during the passage of the carrier through the tube.

The first carrier used in the New York system was made of sheet metal bent into a cylinder and riveted,—the convex bottom piece being riveted to this shell in an overlap. Later, weight was saved as well as a tighter shell secured by making it of seamless tubing,—the bottom being riveted on as before. This shell is the one still in use, but, as the consummation of experiments begun over a year ago, carriers are now being constructed in one piece, cylinder and bottom, being spun up from the sheet metal. The packing rings, however, are still riveted to this one-piece shell, which permits of danger from the dropping of loose rivets into the tube, as well as from the chance given to moisture to enter the carrier through the piercing of the shell. To obviate both these disadvantages and to secure what has not yet been achieved,—a perfectly water-tight carrier,—plans are now being drawn for an improved carrier designed by one of the officials of the company.

Buffers. In the early carrier these are of heavy felt, having strips of sole leather riveted together to form the wearing face. Later a buffer made of small pieces of cork, moulded into a form, was used, and finally it was found possible to make this body of solid cork, which is now the buffer in use. During the experiments with buffers, one of leather stuffed with horse-hair was tried, but the leather covering was liable to rip, and the loose hair released into the tube became a sure means of wedging carriers and of thus blocking the line.

The solid cork buffer now used is faced with strips of sole leather and is tapped through the centre for the bolt fastening it to the carrier bottom, and for the brass stud piece holding the metal disc used in making contact at the intermediate machines.

Lids. The early lid, and the one still used on the smaller New York circuits consists of a flat piece of metal with bent edges, and having three radial bolts locking through holes in the shell and the bent edges of lid. These bolts are operated eccentrically by a lever or handle on the top of the lid. The lever is also placed eccentrically so that when swung into the locking position, it entirely clears the tube. In transit, the walls of the tube prevent the lever from swinging around and thus unlocking the lid. The necessity for a positive locking device is apparent from consideration of the result that will ensue if a carrier containing mail comes open in the tube. The mail is allowed to enter the tube, and may be ground to bits by the following carrier.

The three bolt lid was at first tapped with two holes, copying that of the smaller carrier of the six-inch Philadelphia system. These holes served for the equalization of pressure inside the carrier and the tube. In the eight-inch systems, these holes had to be closed up as well as the large opening at the hinge, in order to prevent the entrance of water of condensation, a feature of the longer lines and higher pressures. On the long Grand Central circuit, this precaution was not found sufficient and it was necessary entirely to re-design the lid. The device now in use is a metal dish setting tightly inside the carrier against a rubber gasket. It is also an attached lid, having a two-leaf hinge. This hinge is beneath the lid, while the locking mechanism consisting of four bolts, entering the carrier shell at four different points, is on top but lower than the outer rim of the carrier, for protection against injury. The bolts are thrown by a lever as in the case of the three-bolt type, and the lever is kept from being bent upward and thus allowing the lid to open by a metal spring which clutches it firmly in place after locking. The rubber gaskets make this carrier somewhat difficult to lock, and at times the bolts have been shot over the top of the carrier instead of through the holes in the shell without the operator being aware of the mistake.

Packing Rings. The outside diameter of the bearing rings is $8\frac{3}{32}$-inch when newly packed. In use, they are allowed to wear down to 8 inches, or $\frac{1}{32}$-inch below. The packing material consists of a combination of rubber and duck, vulcanized as hard as is possible without impairment of the tissues. Formerly the rings were of cotton fibre and easily wore out, lasting only 1,500 miles per carrier. The wear now is nearly 9,000 miles. This wear of packing rings is the largest item of maintenance cost. The weight of the carrier makes a great difference in the life of the rings, and the condition of the tube interior is also an important consideration. Previous to the cutting of the long Grand Central circuit, its tubes were at times drenched with moisture. This acted as an efficient lubricant, but after the tubes dried out their interior became so gummy that it was necessary to send oil through the line in a specially prepared carrier. The best normal lubrication is on the two shorter circuits,—the best of all, probably, being on the Brooklyn line where enough oil enters the expansion joints on the Brooklyn Bridge to keep the tube in a moderately greasy condition, but not enough to rot the packing.

The location of the packing rings on the shell of a carrier is determined (after the length and diameter of the carrier has been decided upon) by the ability to pass the brass bends of 8 ft. radius. The rings are placed so that, in going around a bend, the centre of the carrier just escapes touching the inner or shorter side of the brass tube, while its ends are almost in contact with the side opposite. On the 8-inch carrier the rings are 3-inches from the ends, although this could be made a trifle less with safety, if all the bends were absolutely uniform.

Maintenance of Carriers. In the long New York tube, water causes the rusting of the carrier and a consequent shortening of its life. Its entrance is principally under the packing ring nearest the front or buffer end where it causes a rusting that eventually eats through the shell. Various preventives have been used, as painting the shell under the ring with a waterproof paint used in electrical construction, and galvanizing or tinning the entire shell body before the carrier is made up. Under the tinning process, the shell became softened from overheating, and would buckle easily by the ordinary impact of operation. Galvanizing has finally been adopted, but is not altogether satisfactory with a very wet line.

In addition to the shortening of a carrier's life by rust, the weight and consequent momentum of this rapidly moving shell subjects it to

severe wear and tear. Carriers receive their hardest knocks in collisions or in blocks, but even ordinary operation furnishes a strain at each receiving station despite the use of buffers and the methods to check the carrier's velocity.

The racking of the carrier received in operation, its deterioration from rust, and the replacing of the bearing rings after wear constitute a charge on the New York circuits of 15% of the entire operating expense. On the simple Bridge circuit, this maintenance cost is only 7%. This abnormally heavy charge will be decreased by the one-piece shell, and by further developments in packing material. At present, however, it keeps a considerable force busy at the Company's repair shop situated in its power house, near the northern terminal of the Grand Central line.

THE AIR CURRENT.

It is not the intention of the writer to discuss here the general principles underlying the flow of air in tubes. The theory on the subject is given in the regular college course, and the more important relations existing between pressure, expansion, velocity and temperature are well known.

Pressure and Exhaust Systems. In pneumatic tube transmission in Europe the two systems of propulsion, compression and exhaustion, are ordinarily used together. Thus in the London tubes, compression is used in the outgoing line and exhaustion in the incoming; and the store systems adopt the same arrangement, the great advantage of which is in the concentration of plant at a central station. In power required, the exhaust system is the more economical, for in all cases, most of the work is done in moving the column of air through the tube against the friction of the interior, and this friction varies directly with the density of the air.

In all the large tubes the compression method alone is used. It affords better air cushions, allows the operating cylinders of the terminal apparatus to be made smaller, and, if leaky joints exist in the tube, keeps water and gas from entering. Air at a constant initial pressure is forced in at one end of the tube and kept supplied by the compressor as rapidly as it flows off. When the current reaches the open end of the tube its pressure has fallen to atmospheric and its velocity has materially increased, due to the expansion of the air. The rate of increase in the velocity is not uniform, being greater as the end of the tube is approached. A constant air current is maintained instead of the intermittent flow of some of the small systems where carriers are dispatched only occasionally.

Efficiency. This continuous air current is compared by Mr. Batcheller—very aptly—to a rapidly flowing stream of water and the carriers to boats borne on the surface of the stream, the velocity of which is but slightly influenced by them, so that—except for a slight drag—they move almost as rapidly as the water which carries them. Similarly with the air current in a horizontal tube—except for a slight friction against the sides—the carrier's velocity is that of the air. The power is really expended in keeping the air current in motion, and the presence of carriers is of little moment. The efficiency of such a system is therefore seen to be very low if power alone is considered; and it may be said in passing that this factor restricts its use to certain definite quick dispatch functions.

As a matter of fact the difference in pressure between an empty and a full line on the long New York circuit was from 2 to 6 pounds with the pressure 11½ pounds for the tube without carriers. The larger difference was rarely reached, only at extraordinarily dry conditions of the tube interior. Usually, the presence of each carrier makes a difference of less than two ounces in the pressure. The compressors are kept at a constant speed to give a fixed volume of air, and the pressure fluctuates with the loading of the line. With the fixed air supply, the velocity of the current at the open end of the tube remains the same. At the first part of its course, however, the speed is slower when the tube is heavily loaded, and between a full and an empty line the difference in *average* speed is about 10%.

Compressors. On small tubes and with low pressures, blowers may be used to furnish the air current. On the large mail lines, with pressures ranging from 5 to 15 lbs. (varying with the length of line) to give a speed of 30 miles per hour, power is furnished by air compressors. These act practically as blowers, keeping a constant supply pumped into one end of the line and allowing it to flow off at the other.

Three kinds of air compressors are used on the New York system. The Rand, the Ingersoll-Sergeant, and the compresscr made by the Philadelphia Engineering Works. These are all of the duplex type, compressing to low pressures under slow speed. They are direct connected to Corliss engines, having non-condensing cylinders. In the case of the Rand compressors at the Brooklyn Post Office, a speed of 73 revolutions per minute gives six pounds pressure and an indicated H. P. of 46.8. The theoretical delivery at this speed is 1,500 cubic feet of free air per minute,—the actual delivery being about 1,400 cubic feet. The stroke of this compressor is 20 inches; the diameter of its air cylinders is 24 inches, and that of the two steam cylinders,

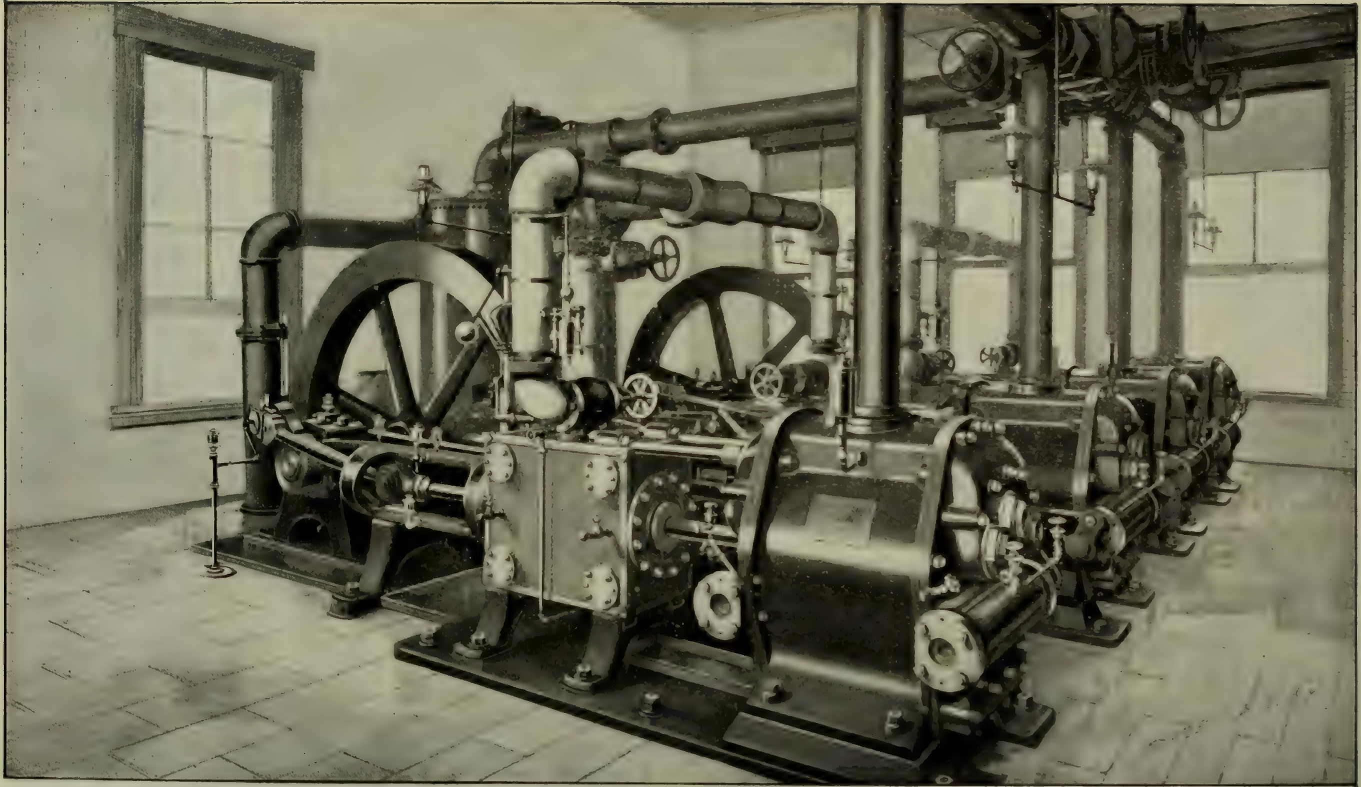


FIG. 9. RAND COMPRESSORS AT POWER HOUSE "A."

12 inches. The air cylinders have Corliss inlet and special poppet outlet valves with springs so light that little pressure is required to open them. The air valves of all three types of compressor make considerable noise and clattering, but the Rand is probably the quietest.

On the complete system are two compressors at the Brooklyn Post Office, four at the New York Post Office, three at "Madison Square" station, and two at the power house on Depew place. (See Fig. 9). The Company makes its own steam at the last named plant, but at all the others the supply is obtained from outside sources.

The Course of the Current. In an ordinary circuit with a compressor at either end, air flows from one station through overhead piping in the building, passing the transmitter on its way, and thence out into the underground tube. At the other end of the line, it is deflected at the open receiver, passing down into the engine-room of the station and into a large iron tank open to the atmosphere. There is a tank at each station, and the compressors pump from them so that the same air is used over and over again except for the fresh supply at the open valves of the tanks to compensate for losses. These losses are those of leaky joints in or out of the building, and those at the terminal machines, and constitute in New York about 10% of the air current. The tanks act also as separators for the reception of oil, dirt and moisture from the line.

The current must be continuous, and only carelessness on the part of the engineman will account for the dropping of pressure and a consequent catching up of carriers in the tube so that a block may occur.

Oil Leaks. These have been a somewhat interesting incident of the New York power plants, occurring in the air piping between the compressor and the transmitter in the same building. It was found that the air under pressure and containing minute particles of oil carried over from the compressor, renders the securing of tight joints extremely difficult. Before this fact was first brought to attention by the persistent though slow drip of oil in the early engine rooms from the joints overhead, only ordinary care was observed in making up with red lead the screwed joints of the wrought iron air pipes leading from the compressor. To stop these leaks, a number of patent joints were tried, as well as rusting the joint and the ordinary methods used in such cases, but all without much effect. Now, when the joints are made, they are screwed up as hard as possible with shellac, and while this is not entirely effective, it is practically so. The high temperature of the air as it leaves the compressor, and the fact that the oil is vaporized and under pressure is the reason for the above experience, which is to some extent a novel one. An experiment was also tried in the use of spiral riveted pipe instead of the ordinary wrought iron, but despite the maker's guarantee, it was found to leak oil.

Various kinds of gaskets were tried for the flange joints, including the ordinary soft packing and a metal combination. Jenkins' '96 has been found the best in the resisting of the heat and oil. For the engines full metallic packing has been adopted.

Temperature Conditions. Because of the low pressures used, the air cylinders and piping are not jacketed, and the air consequently is heated higher than the theoretical temperature of compression. On the Brooklyn line, with six pounds pressure, this temperature is found to be 150° at the compressor, and, 350 ft. away (150 ft. of which is underground), it is 110° to 115°. From this point the temperature falls rapidly until it becomes that of the ground. This drop in temperature means a deposit of the moisture contained by the air, which is considerable on a day when the humidity is high. The condensation here described is a serious problem, and its bearing on the transportation of mail will be seen further on.

OPERATING EXPERIENCE.

So far as ordinary operation is concerned, the problems are those of any system of transportation. But in the early days of the New York lines, there existed special conditions arising from the total lack of precedent, and, in the nature of the system itself, there will always be the peculiar features of inaccessible transit from dispatcher to receiver, and the handling, under high velocities and out of sight and control, of so important a commodity as mail matter.

Care in Handling Mail. The precautions necessary in the preservation and constant expedition of the mail have built up a force of operators trained in the special knowledge required. They are, to some extent, factors in the post office system—a transportation problem, by the way, the size and importance of which is appreciated by few outsiders ;—and, in the more mechanical side of their work, must be able to make ordinary repairs of the apparatus in their charge, and to handle minor accidents and emergencies pending the arrival of one of the mechanics of the Company. Their care and alertness is constantly called for in the locking and inspection of each carrier before it goes into the tube, for a carrier opened in transit almost surely means the practical destruction of its contents, and a complete shutdown of the line for some hours. Early in the Company's history,

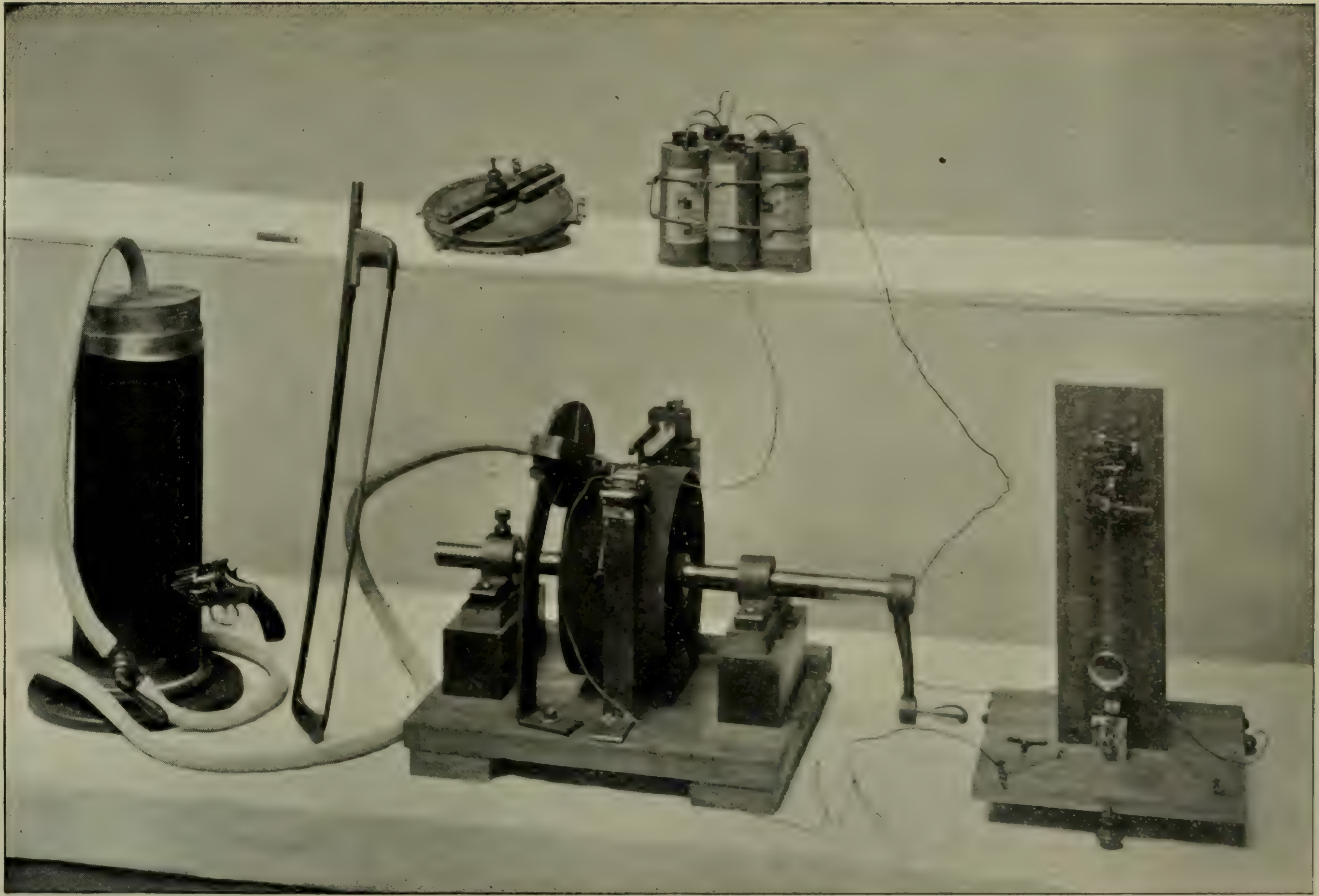


FIG. 10. CHRONOGRAPH USED IN PHILADELPHIA FOR LOCATING OBSTRUCTIONS IN PNEUMATIC TUBES.

and with green men, some such instances did occur—most of them during the operation of the tubes by employees of the Post Office—but there have been none within the past 18 months. Of course, the locking of the carrier should be made automatic as is the rest of the system, and the new design of carrier now being developed will, it is hoped, furnish a practical means of preventing its entrance into the transmitter unless locked.

Blockades. The system is comparable to a cable railway in its occasional shut-downs. Rarely these may be laid to an employee's incompetency, to his losing his head, or to a lax inspection by him of the carriers and apparatus under his control. With the present operators they are nearly always caused by breakage or failure to act of the terminal machinery, and by the introduction of foreign matter, principally loose carrier parts, into the tubes. Blockades have become more and more infrequent with the improvement in machinery and carriers, but, in the nature of the system, some must always be anticipated and, when they exist, must be handled with the same degree of preparedness and urgency as those of street railway transportation.

Manner of Clearing. With the blockade once established, pressure is reversed and run up as high as can be obtained,—a trifle over 30 lbs. on the long line. In ordinary operation, the Grand Central circuit had, at times, 35 carriers in the tube between terminals ; in the event of a block this number has been increased to 50 or even more, making the task of starting them a difficult one. Usually the blocked train would release suddenly and come back to the originating station with tremendous force, from which the apparatus and employees had to be protected.

Location by Sound. In case a blockade could not be moved under pressure it would have to be located and the tube opened,—something happily, that has never occurred in the regular operation of the New York circuits. Where a line is equipped with gage pipes, 300 ft. apart, say, difference in pressure will locate the carrier between two of these, and the plan of the tubes will usually show a bend or likely weakness that may point to the exact position.

In the construction of the Philadelphia six inch circuit, no gage pipes were used, and a case occurred where an obstruction had to be located in order to excavate. For this, Mr. Batcheller made use of sound waves, employing a rough chronograph (shown in Fig. 10), modelled after the device of a French investigator for use on the small tube lines. A pulley served as the drum, its surface being smoked to record the time measures of a pendulum and, for further divisions of a second, a tuning fork. Firing a pistol into the closed tube, generated sound waves which were deflected from the obstruction and returned to the chronograph. This recorded the interval, to a thousandth of a second, between the firing and the echo ; and the velocity of sound at the temperature of the ground, gave the distance to the obstruction. The average of five tests fixed the location within ten feet. Mr. Batcheller has since improved upon this rough apparatus and now has a simple and effective means of locating bad obstructions.

Troubles From Water. The extent of the deposit from moisture of condensation was not anticipated by the inventor, and after the wetting of mail began in the early days of operation, everything possible was done in the way of temporary relief, pending the design and manufacture of tight carriers. In addition, various methods of abstracting the moisture from the air before sending it into the tubes were investigated, but none reasonably inexpensive were found ; and up to the time of cutting the long circuit, in the fall of 1899, reliance had to be continued in making the carrier as waterproof as possible.

With such a carrier, and one thoroughly protected against rust and deterioration, a moderate amount of water in the tubes is of no harm, and furnishes an efficient lubrication. On the short Produce Exchange and Brooklyn circuits little water is observed, and a fairly tight carrier is all that is necessary. Since the converting of the long Grand Central circuit into two short ones, this is also true of it.

Conditions on Grand Central Circuit. Under former conditions, however, considerable trouble was experienced. The worst wet spells occur in winter during a warm wave just after a freeze, and in summer on a hot and humid day. When the air and ground are of about the same temperature — for weeks in the fall—the tubes may be so dry as to require lubrication. But during the wet seasons, the circuit was sometimes drenched, water trickling out of the intermediate machines and, to a less extent, at the open receiver, while the carriers looked as if they had been dropped into puddles. Carriers, even while tight originally, might under such extreme conditions, betray a weakness obtained in the hard racking at the various machines, and permit of the entrance of a minute spray of water under pressure. In a few cases of bad defects in the carrier or of the cracking of its shell under the packing ring, a considerable amount of water reached the interior of the carrier, causing severe wetting of the mail matter contained. In no instance, however, has mail been so damaged that it cannot be dispatched to its destination after drying out, and in nearly every case of the so-called wetting, the extent of the manifestation has been a

a block from any cause, was invariably made more troublesome and longer in duration because of the branches and their switches.

PRESENT GRAND CENTRAL CIRCUIT.

Operation of Intermediates. For the foregoing reasons, therefore, the circuit was cut in two at "Madison Square" station, which was made a terminal for both short circuits. Of these the longer (the south division) is only two miles from the G. P. O. to "Madison Square", and there is but little more moisture experienced than exists on the model Brooklyn circuit. It has as an intermediate station, branch "D", while "F" is an intermediate on the north half. A short circuit with but one intermediate is comparatively simple as regards operation. The two original machines, provided with some minor improvements, are used at each station, but only one at a time is operated, the other being in readiness in case of trouble. At station "D" this method of operation is greatly facilitated by the nearness of the switches, which are just in front of the building, 100 feet from the machines. This permits of either machine being instantly thrown in or out of use by the sound of the carriers passing. At station "F" the switches are on 4th Avenue, 1,400 ft. away, and the passing of carriers is given by an electric indicator at the top of the tube ringing a bell in the station.

Re-dispatching Carriers at Madison Square Sta. "Madison Square" station is equipped with a power plant and, for each short line, a combination open receiver and transmitter. Carriers in either direction come out at the station and are there re-transmitted. This is done quickly and easily by having the receiver of one terminal connected with the transmitter of the other, and by the use of a vertical transmitter leading off from one corner of each receiver pan. Carriers are thus redispatched without lifting them from the pan into which they roll from the open receiver. The open receiver, with some minor changes in the method of opening and closing the sluice-gate, is similar to the one in use on the rest of the system.

Vertical Transmitter. The vertical transmitter is from designs by the Superintendent of Machinery of the Company, Mr. B. H. Blood, M.E., Cornell, '89. It is a short vertical section of tube closed at either end by a gate, and leading off from the bottom to the street line. With the upper gate open and the transmitter ready for sending, a carrier is dropped into the sort section, a lever pushed over by hand, closing the upper gate and opening the lower one, and thus allowing the carrier to drop out into the air current. As it goes, it trips a fin-

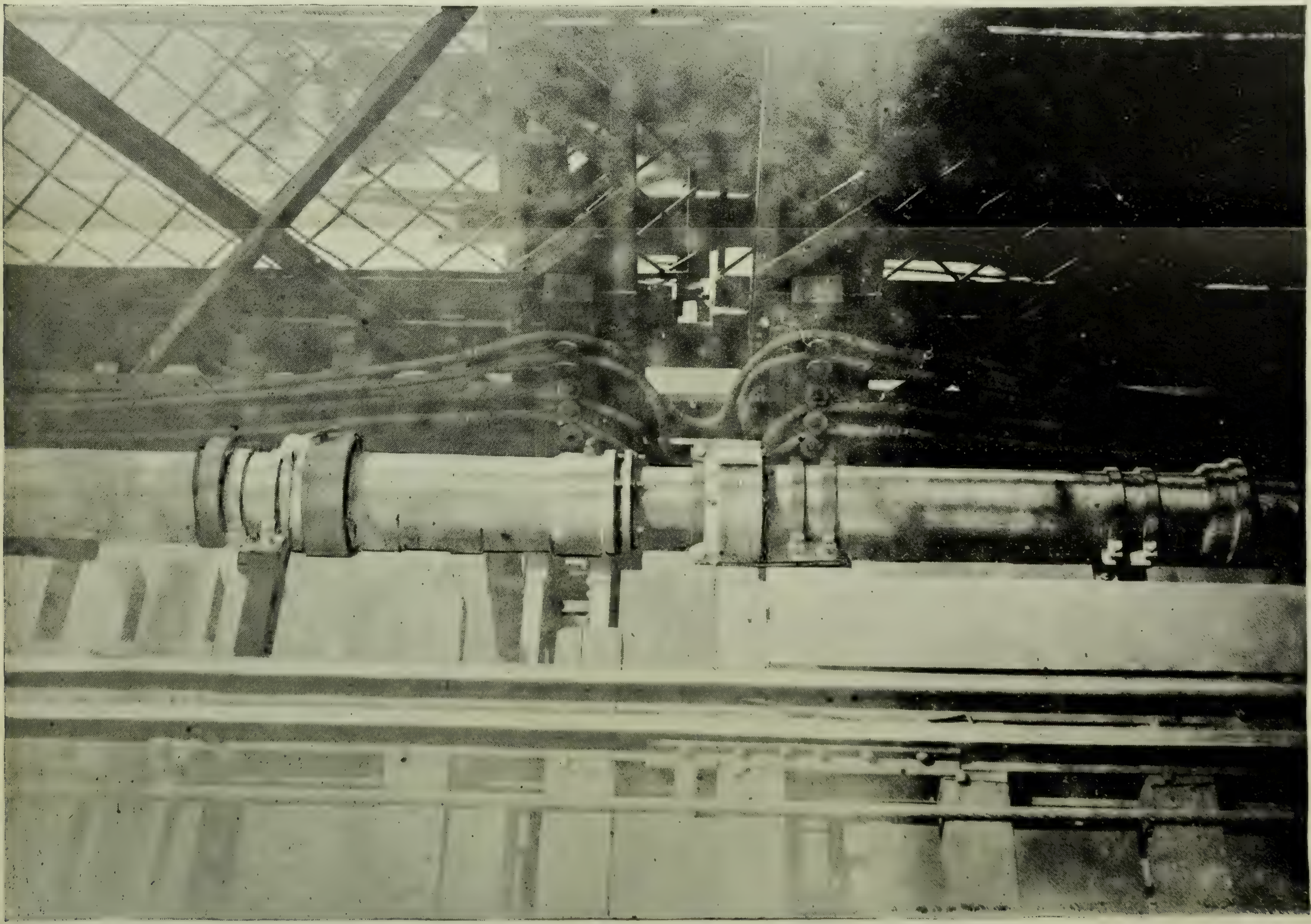


FIG. 12. EXPANSION JOINTS. BROOKLYN BRIDGE.

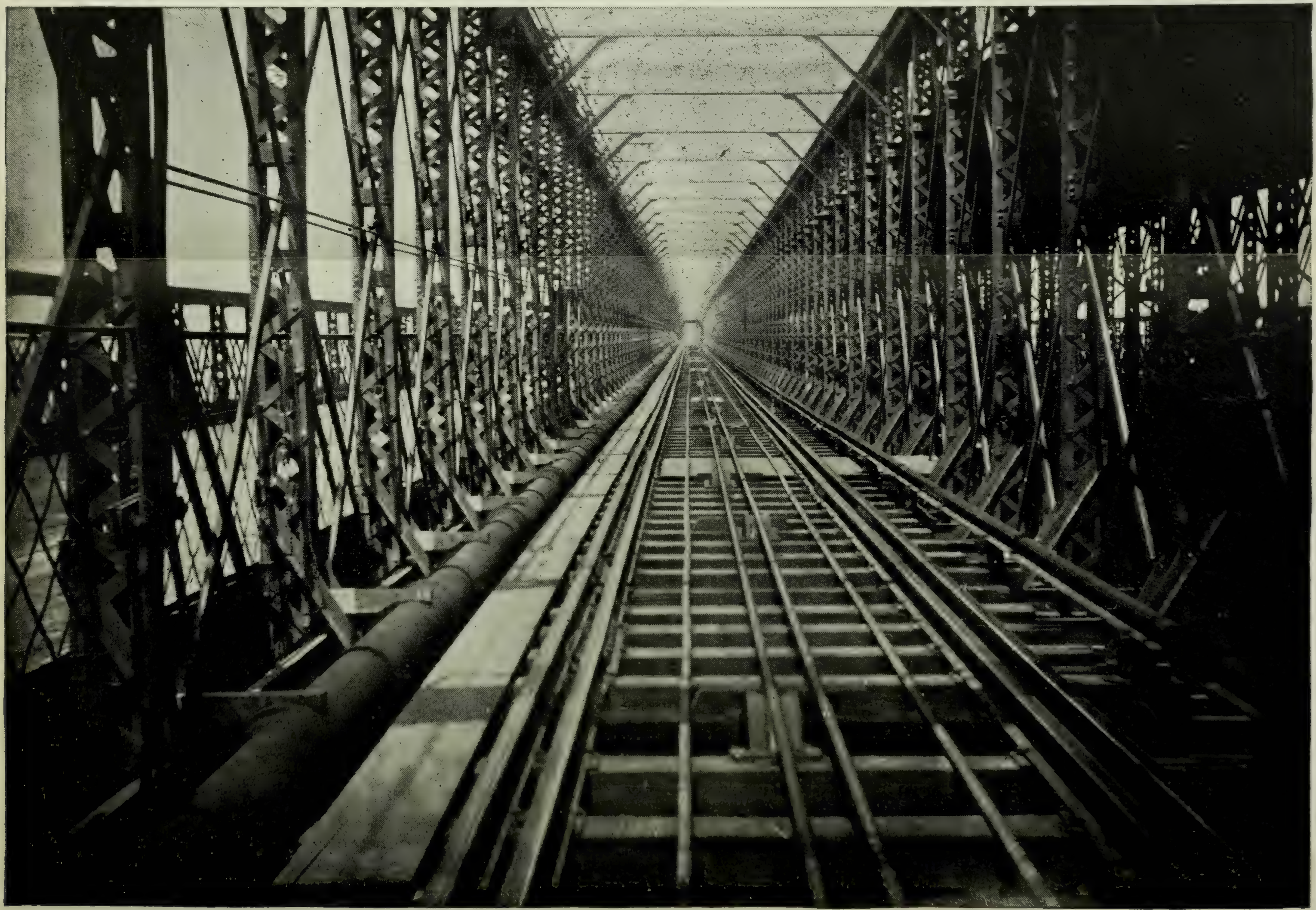


FIG. 13. PNEUMATIC TUBE ON THE BROOKLYN BRIDGE.

ger which reverses the positions of the gates and takes the tube pressure off the short section,—when the transmitter is again ready to receive a carrier.

New Design of Machine for Intermediate Station. The present intermediate machines at "D" and "F" are working well in the manner described in a previous paragraph, but it is thought that they can be greatly simplified by another of Mr. Blood's designs. It is virtually an open receiver with an air lock secured by using the two-gate principle of the vertical transmitter. A carrier can thus be received and taken out at atmospheric pressure, while the air current, under three or four pounds pressure, flows on through a by-pass back of the rear gate. This combination will give all the results of a closed receiver while acting in one-third the time.

With this new intermediate machine it is also intended to do away entirely with the switches, or to use a new design by Mr. Bacheller of a closed switch (sections of tube thrown in and out) located in the basement of the station itself.

LAYING TUBES ON THE BROOKLYN BRIDGE.

Expansion Joints. The bridge portion of the Brooklyn circuit was not constructed until the summer of 1898. The work was done under the writer's supervision, with Mr. Bacheller as consulting engineer, and presents some original features which may be of interest here. The tubes are laid exposed on each interior trackway, and the expansion in these long exposed stretches is, of course, considerable. The movement of the bridge itself is a maximum of 14 inches at the centre, and of ten inches on each span. This is taken care of, as well as the difference in expansion between the wrought iron of the bridge and the cast iron of the tubes, by three expansion joints on each tube line. In addition there is an expansion joint on each line at the Franklin St. bridge of the New York approach, and two plain up and down slip movements at each abutment where a large vertical deflection is noted as the heavy bridge trains pass.

The expansion joint consists of a large stuffing box in which a brass tube slides back and forth. The six on the bridge structure permit also of a lateral and a vertical motion by the use of a ball and socket.

Supporting the Tubes. On the bridge structure the tubes are strapped to the ties. After leaving the structure they are anchored solidly at the abutment, and also at the end of the approach where they go under the walkway. Between these anchors the tubes are supported on cradles provided with rollers, which allow of a creeping back and forth.

After completion, the lines were tested under 30 lbs. pressure, and their maintenance since has been negligible. Occasionally a slight leak at a joint will be noticed, due usually to the vibratory motion of the structure caused by passing trains, but these are easily caulked tight.

Difficulties of Construction. The work was performed under difficulties because of the heavy and almost constant traffic. In the middle of the day some caulking could be done, great care being exercised in giving warning of approaching trains. The third rail and the shoe of passing motor cars were equally sources of danger. The hauling and placing of the tubes, and most of the jointing was performed at night from 11:30 p. m. to 3:30 a. m., when by special arrangement with the bridge authorities, a train was shunted back and forth on one track, leaving the other free for transporting the tubes on flat cars and the concentration of work on them. Despite every precaution, two regretable accidents occurred—one in which a workman was killed, and in the other a caulker seriously injured.

MAIL MATTER HANDLED.

Statistics of Operation. In amount of mail transported, the Produce Exchange is the least important of the three circuits. About 630 carriers are dispatched in each direction on an average day, 903 being the highest record. The circuit is operated continuously from 5 a. m. to 9 p. m. of each week day. The possible carrier interval is ten seconds, giving 5,760 carrier trips as the tubes' capacity.

On the Grand Central circuit the trips average a trifle over 2,000 per day, or only 50% of the line's capacity under the 15 seconds headway observed. The circuit is operated from 4:00 a. m. to 9 p. m.

The Brooklyn line transmits about 1,000 carriers in each direction daily during the working hours from 4:00 a. m. to 11:00 p. m. The possible carrier interval is 12 seconds, or a capacity of 5,700 carrier trips.

Quantities Cared for. The number of pieces of mail matter transmitted on the three circuits is shown by the following record of one day's work : on the Produce Exchange line, 98,599 letters and 22,487 pieces of papers, merchandise, and circulars ; on the Grand Central circuit 1,019,647 letters and 91,722 pieces of the lower classes ; and between New York and Brooklyn, 113,910 letters, and 61,024 pieces of other kinds. The first and the last named circuits handle practically all of the letter mail originating during the hours of their operation ; the Grand Central line in the same time, cares for 85% of its letter mail, the remaining 15% representing pouched mail in transit through the city, and considerable quantities of lower class matter. The proportion of lower class matter handled by all the circuits combined is but a small proportion of the total bulk of that matter to be transported, but the only excuse at all for caring for even such limited amounts is the desire to utilize the idle moments of the tube service.

Supplementary Dispatches. One of the important developments of the pneumatic tube system is observed in the late (or supplementary) dispatch of small amounts of railway letter mail posted after the regular time of closing the dispatch. In many cases letters are thus enabled to save considerable in the time otherwise required to reach their destination. What some of these savings are may be seen from the following table taken from post office reports :

STATEMENT OF TIME GAINS IN SUPPLEMENTARY DISPATCH VIA TUBE.

Time of first carrier.	Time of last carrier.	Estimated number of letters.	Number of carriers.	Closing time.	Time gained in dispatch of mails from New York, N. Y.
8.05	8.09	175	2	7.30 a. m.	4 hours 15 minutes.
8.09	8.09	54	1	7.30 a. m.	3 hours 58 minutes to 6 hours 56 minutes.
9.28	9.28	10	1	8.30 a. m.	2 hours 59 minutes.
9.48	9.50	90	2	9.00 a. m.	2 hours 30 minutes.
10.12	10.12	30	1	9,30 a. m.	2 hours 2 minutes.
10.49	10.49	52	1	10.00 a. m.	1 hour 30 minutes.
11.14	11.14	175	1	10.30 a. m.	1 hour 2 minutes.
11.15	11.15	80	1	10.30 a. m.	1 to 5 hours.
12.22	12.23	700	3	11.30 a. m.	4 hours to 7 hours 15 minutes.
12.24	12.25	400	2	11.30 a. m.	1 hour 58 minutes to 2 hours 58 minutes.
12.55	12.55	15	1	12.00 m.	1 hour 52 minutes.
1.20	1.22	277	1	12.30 p. m.	1 hour to 2 hours.
1.22	1.23	17	1	12.30 p. m.	1 hour 40 minutes.
2.17	2.17	100	1	1.30 p. m.	1 hour 56 minutes.
2.21	2.23	900	3	1.30 p. m.	1 hour.
2.20	2.20	150	1	1.30 p. m.	2 hours.
2.44	2.44	40	1	2.00 p. m.	1 hour 53 minutes to 2 hours 8 minutes.
2.49	2.49	80	1	2.00 p. m.	15 minutes.
2.52	2.52	50	1	2.00 p. m.	5 hours 29 minutes.
3.20	3.22	600	3	2.30 p. m.	2 hours 24 minutes to 6 hours 59 minutes.
3.23	3.25	900	4	2.30 p. m.	2 hours 25 minutes to 11 hours 50 minutes.
3.55	3.55	50	1	3.00 p. m.	32 minutes.
4.15	4.18	700	3	3.30 p. m.	1 hour to 3 hours 15 minutes.
4.20	4.20	100	1	3.30 p. m.	1 hour to 5 hours 59 minutes.
4.22	4.22	100	1	3.30 p. m.	11 hours 58 minutes.
5.10	5.12	700	4	4.30 p. m.	2 hours 15 minutes to 3 hours.
5.41	5.45	600	4	5.00 p. m.	2 hours 35 minutes.
8.18	8.28	3,200	14	7.30 p. m.	6 hours 35 minutes to 11 hours 45 minutes.

FUTURE OF PNEUMATIC TRANSMISSION.

Present Contracts. The amount of the present tube contract for Manhattan is $158,500 per annum, which includes a labor allowance of $10,000, In the Brooklyn line it is $20,200 per annum, also including a labor allowance of $6,200. Wagon transportation for Manhattan costs the Post Office Department $239,200 annually, which is made up of $121,800 for the mail messenger service, and $117,400 for the mail station portion. This latter is a rate of 20.14 cents per wagon mile (1,733 miles daily),—while the Manhattan tube service costs the post office 3.17 cents per carrier mile (15,431 miles daily).

Wagon and Tube. These figures are not intended as a comparison, for bulk is not taken into account by them. In fact, there can be no comparison between wagon and tube transportation by bulk, as the formor will always be much the cheaper. It is in speed, seconded by continuous and unobstructed transmission, that the pneumatic tube advocates find their strongest arguments.

It is not the purpose of the writer to discuss in this paper the future of the tube service or its relative economy as a system of mail transportation for cities. He believes, however, that it is worse than useless to compare it with the entirely differing method of surface handling in bulk and at separated intervals. The latter must always be the most economical and practically efficient means of caring for the heavy mail of the 2d, 3d, and 4th classes which neither demands nor receives expedition. The low power efficiency of pneumatic transmission and its consequent high cost, restrict its uses to the quick and continuous forwarding of first class mail, telegrams, messages, special delivery letters, and possibly, the smaller and more important express matter and parcels, and the handling in limited amounts, of special city newspaper deliveries.

Future Possibilities. In the writer's opinion the future system for an entire city will be one of tubes ranging in size from three to eight inches—possibly to twelve inches, if the handling of certain kinds of merchandise is found to pay—and reaching to all parts of the city in conjunction with an auxiliary service by automobile and street car for the lower classes of mail matter. In such a system there will undoubtedly be more small than large tubes; and the concentration of operation and of power will permit of an elastic dispatch in varying quantities at a much lower cost per carrier mile than the Post Office Department is now paying.

Long and Large Lines Impracticable. The pneumatic tube is essentially a system of urban dispatch, and one of its uses is to take up and continue the transportation of letters where the railway leaves it off. It is absurd to speak of tube lines between widely separated cities, or to mention them as possible competitors of the steam railway. It is equally impracticable to talk of pneumatic tubes of large diameters. Such are commercially impossible.

Conclusion. It has not been possible in the scope of this paper to go into the relations of the pneumatic tube to the mail transportation problem of large cities. To do so would necessitate a discussion at some length of the present post office methods (an extensive and complex subject in itself) and the changes in them necessary to the most efficient use of a complete tube system. It is an attractive field, however; and, to young engineers, there can be no more interesting transportation problem than that involved in the handling of United States mail, both on the railway, in the city, and at the post office and its branches.

If this paper is already too long, the writer's only excuse is that the subject is one new to most American engineers; and so little written knowledge about it (and that in widely scattered articles and transactions) exists, that methods and apparatus are equally apt to be unfamiliar to the general reader. To those who are interested, and who wish to go further into the technical side of the system, an inspection of the practical operation is advised. Mr. Blood will be glad, at any time, to show Cornell men over the New York lines; in Philadelphia, also, the superintendent, Mr. Kenneth Stuart, is a Cornell graduate.

The writer's acknowledgments are due to the officials of the Tubular Dispatch Co., and the Bacheller Pneumatic Tube Co. for their courteous co-operation in the preparation of this article and for permission to use the plates shown.

ECONOMICS OF STREET PAVING.

(LECTURE BEFORE THE COLLEGE OF CIVIL ENGINEERING OF CORNELL UNIVERSITY BY S. WHINERY, APRIL 27, 1900.)

The importance of street pavements as an element in municipal public work and municipal finance seems to be not fully appreciated. This branch of municipal public work has not received from municipal engineers and city officials the attention and study that its importance merits and demands.

Projects for sewerage and water supply systems are not usually undertaken without the most careful study and consideration. Elaborate surveys are made, the character and capacity of the work fully discussed, the present and future needs of the community are considered, and the financial aspects of the enterprise are carefully weighed and provided for. Not infrequently the best expert counsel is consulted in order that no mistakes be made, and that the problems presented may be solved in the best and most economical manner. Moreover, the general principles underlying these subjects have been worked out theoretically and verified by observation and experiment, the necessary data having been carefully collected and recorded. It is not so in the department of street paving. This work is often, if not generally, entered upon by municipalities without adequate investigation or consideration. When it is decided that a street is to be paved, the question of the kind of pavement to be used is decided by consulting the preferences of the people owning property or living along the street. These preferences are usually founded on sentiment or preconceived notions, without much reference to the scientific or economic principles involved. Too often the question of first cost, regardless of real economy, is allowed to out-weigh all other considerations. The unsatisfactory results can be seen in many streets in nearly every city.

It cannot be claimed that this condition of affairs is the result of the relative unimportance of street paving as a branch of municipal improvement. Whether measured by their influence on commercial prosperity or by the amount of money invested in them, street pavements belong in the first rank of importance. It is probably because of the failure of the public to appreciate their importance that the statistics of street paving are meagre and incomplete. Until quite recently no serious attempt had been made to collect and publish such statistics for American cities. Something in this way was attempted in the Eleventh Census (1890), but the information secured was very incomplete and unsatisfactory. More recently the Bureau of Labor has, by direction of Congress, taken up the collection of statistics of municipal works, and the first results are published in Bulletin No. 24 of the bureau, issued in September, 1899. In this report are given some statistics of street paving in American cities that add materially to our knowledge of the subject. The examination embraced 139 cities, being all but one of the cities of the United States having a population of 30,000 or more. The tabular statements give the number of square yards of the various kinds of pavement in each city, but no statistics of their cost. We may arrive at an approximate estimate of their cost, if we assume for the purpose the prevailing average prices paid for such work during the past ten years. In round numbers the total number of square yards of each of the principal kinds of pavement and their approximate cost is as follows :

82,126,000	sq. yds.	Macadam road	at 75 cts.	$ 61,594,000
20,600,000	"	Cobblestone pavement	at 80 cts.	16,480,000
34,785,000	"	Granite block "	at $3.50	121,747,500
29,238,000	"	Wooden block "	at 1.25	36,547,500
18,868,000	"	Brick block "	at 2.25	42,453,000
31,305,000	"	Asphalt & Asphalt block	at 2.75	86,088,750
102,029,000	"	All other kinds	at 1.00	102,029,000
318,951,000		Costing		$466,939,750

It will thus be seen that these pavements have cost over $450,000,000, and it must be remembered that this first cost of construction does not represent their full actual cost, since many of them have been renewed one or more times, and a very large sum has doubtless been expended upon their repair.

The population of these 139 cities is given in the same Bulletin as about 20,000 000. The first cost of these pavements was therefore nearly $23 per capita on the present population of these cities.

It thus appears that the item of street paving is of such actual and relative magnitude in municipal improvement as to deserve the most careful attention and systematic study.

The investigation should not only embrace paving materials and pavement construction, to which branch of the subject most attention has been heretofore directed, but it should extend to the commercial and economic value of street pavements in general, and to the relative value under various conditions of the different kinds of pavement in use.

It is to this branch of the subject which, for want of a better term, may be called the "Economics of Street Paving" that I invite your attention to-day. In attempting to make an economic study of the subject, we are met at the start by the absence of much of the data necessary for even an elementary discussion. A part of this data must be obtained from complete statistics in detail of street travel and a part from carefully conducted experiments to determine the various coefficients which enter into the problem. I can therefore do little more than to indicate the line of investigation, and the method of arriving at results, using, where necessary, approximate and even assumed data. In this way I shall endeavor to point out the various kinds of information necessary for solving the problems, hoping thus to direct your attention to the value of these data, and the necessity for collecting and recording them.

In attacking any problem whose solution is dependent on the results of observation and experiment, it is desirable to determine in advance, as far as possible, the extent and character of the data required, so that we may direct our efforts to obtaining such data without wasting time on unnecessary or unimportant elements or details.

Many of you will doubtless find your professional vocations in municipal engineering. It is a field offering ample scope for your best ability, and unusual opportunities for accomplishing good, by promoting the health, happiness and commercial success of that very large part of the population of this country who live in municipalities. If I shall succeed in calling your attention to the importance of collecting, in your work, the data necessary for an intelligent and scientific discussion of the economics of street paving, and in suggesting some lines along which the investigation may be profitably pursued, I shall have accomplished all that I can now hope to do.

When any public improvement involving the expenditure of large sums of money is proposed, the first question asked by the average citizen and tax-payer is, "Will it pay?" It is a proper and pertinent question, and deserves a serious and business-like answer. It is the question every good business man asks about any commercial or industrial enterprise in which his interest is solicited. And while it is true that there are other considerations that may properly be taken into account in every public improvement, the crucial test that nearly every project of this character must be subjected to, is that of profit or loss. If it can be shown conclusively that a street pavement will materially benefit the community at large, and its individual members as well, it will commend itself to business men, and will not lack encouragement and support.

Is it possible to present the pavement question in such a light to a fairly intelligent public? And in order to do so, is it possible for the municipal civil engineer to reduce the paving problem to something like the same scientific system that he obtains in some other llines of municipal work? I believe that both of these questions may be answered in the affirmative. This department of municipal work can at least be placed on a basis where random opinion and dogmatic statement need not be mainly relied upon.

The city street has its prototype in the country road. Its function is to facilitate intercourse and traffic between different localities. The volume of travel and traffic will determine the character of the roadway that may be profitably provided. In the country districts where population is scant and human habitations are far apart, the natural surface of the soil, cleared of trees and other obstructions, forms a roadway as good as the commercial conditions will justify. As population becomes denser, and intercourse and traffic increase, it is found profitable to reduce the obstructions to travel, to drain the roadway, and to secure a surface more unyielding than the natural soil.

At a further stage of development, usually in villages and towns, it is found profitable to reduce the gradients and to supply an artificial road surface sufficiently smooth and unyielding to offer still less resistance to the hauling of loads over it, and thereby reduce the cost of traffic. Thus we have the gradual evolution of the macadamized road from the country lane, brought about by the effort to reduce the cost of transportation and thereby increase the profits of traffic. Further development and application of this principle results in the modern street pavement of our cities. The underlying motive from beginning to end is commercial profit. Economically, the building of a costly paved street where the country road was alone justified would have been a business blunder; no less is it a blunder to continue the use of unpaved streets or macadamized roads where the character and volume of the traffic is such that it can be more economically conducted over street pavements of the best character.

This brings us to the first important principle in street paving, which is, that the character and cost of the pavement must primarily be determined by the character and volume of the traffic it is intended to accommodate.

There are other important things to be considered, but these are secondary, and the weight to be given them will depend on the importance we may attach to each. They do not affect the truth of the general statement that, from the economic point of view, the value of a pavement must be measured by the return it will yield upon the money invested in it.

This being accepted we may proceed to inquire into the cost of transportation over street pavements, and to consider the relative value of the several kinds of pavement in common use.

The elements that enter into the problem of transportation over city streets are:

First—The cost of constructing and maintaining the pavement.

Second—The cost of power for conducting transportation over paved streets.

Third—The cost of providing and maintaining vehicles used for transportation.

Fourth—The volume of the traffic to be provided for.

Let us consider these in the order stated.

Fortunately, we have sufficient data to enable us to determine with reasonable confidence the cost of constructing each of the kinds of pavement in common use. We have also sufficient information to predict with rough approximation to the truth the life of the pavement under given conditions of use, and the cost of maintainance during its life. The interest charge is a matter of computation, and we can estimate quite closely the value of the old pavement when it becomes so worn as to make its renewal advisable.

If we represent by—

$L.$ = The life of the pavement in years.

$C.$ = Its first cost.

$M.$ = Cost of maintainance during $L.$ years.

$I.$ = Rate of interest.

$H.$ = Value of the old pavement at time renewal becomes necessary.

$X.$ = The cost of the pavement per year.

We shall have

$$X = \frac{C + CIL + M + \frac{1}{3} MIL - H}{L}$$

which may take the form

$$X = \frac{C + M + IL\left(C + \frac{m}{3}\right) - H}{L} \qquad (1)$$

In this equation M represents the total cost of maintainance during the life of the pavement. Usually the greater part of the amount will be expended during the latter half of the life of the pavement, since the cost of repairs usually increases with the age of the pavement. We may with approximate correctness assume, as is done in the equation, that the whole of M is evenly distributed over the last ⅔ of the life of the pavement, and the interest charge will therefore be equal to the interest on one third of M for the whole period.

The quantity H has been very generally overlooked or omitted in similar formulae, but it is an element of great importance in judging of the comparative economy of the several kinds of pavement. Thus, if a brick pavement is laid on a concrete foundation the first cost of the pavement will be greater, by the cost of the concrete foundation and the additional quantity of excavation required, but the durability of the pavement will be greatly increased and when the brick surface is worn out, the concrete foundation should be as good as when first constructed, and a new brick surface could be substituted at a comparatively small cost ; whereas if the concrete foundation is not used, not only will the pavement be much less durable, but there will be nothing available from the old pavement toward the construction of the new, except the excavation of the foundation.

An inspection of the formula shows that the life of a pavement is the most important factor in determining the cost per year of its construction and maintainance, and it is therefore of the greatest importance that we should know with approximate correctness what the life will be under the conditions to which it will be subjected :

In two pavements the original cost of which is the same and the other conditions are equal, if one has a life double that of the other, the real cost of the one will obviously be double that of the other. We can compare them fairly only by ascertaining the cost of each for a unit of time, which unit may most conveniently be considered the year.

It is pretty well established that the life of a pavement, the materials of which are not subject to natural decay, is determined largely by the quantity of travel to which it is subjected, modified of course, by the destructive action of the elements, but each kind of pavement has a different co-efficient of resistance to wear per unit of travel. These co-efficients have not yet been satisfactorily determined though there is a considerable quantity of information available for the purpose. When these co-efficients are determined, and we have ascertained the tonnage of travel of a street which it is proposed to pave, we should be able to predict quite closely the life of any kind of pavement it is proposed to use, and can then apply the equation given (1) to determine the annual cost of construction and maintainance ; and by applying the same process to the different kinds of pavement we can determine which will be most economical to use from the standpoint of cost of construction and maintainance. It must be remembered, however, that cost of construction and maintainance is only one, and usually a very unimportant one of the elements to be considered in determining the real economy of a street pavement.

Each of the quantities entering into formula (1) will, of course, vary more or less in each individual city, with the cost of materials and labor, the interest rate, etc., and in employing this or any similar formula, we must be careful that the conditions are fairly alike, or the results will not be comparable.

Having the necessary data we may by the use of formula (1) prepare a table showing the relative cost per year of the several varieties of pavement in use in any city and thus determine their relative value on the basis alone of first cost and maintainance. Furthermore, we may by taking the average cost of construction and maintainance of pavements of the several kinds in American cities, compare these pavements in a general way and arrive at an approximate estimate of the cost of each per year for construction and maintainance.

Such a table is here given :

TABLE A.

Showing Cost of Construction and Maintainance of Pavements of several kinds on street having a travel of 100 tons per sq. yd. per day—Rate of interest 6%.

	Granite Block on Concrete.	Granite Block on Sand.	Sheet Asphalt.	Brick on Concrete.	Brick on Sand.	Cedar Block on Concrete.	Cedar Block on Sand.	Cobblestone.	Macadam.
First cost ....C.	$ 3.75	$ 3.20	$ 2.40	$ 2.15	$ 1.50	$ 1.80	$ 1.15	$ 1.00	$ 1.20
Maintainance....M.	.40	.60	.50	.45	.48	.36	.30	.48	1.00
Interest............	4.66	3.06	2.31	1.66	.80	.69	.31	.54	.46
Total cost..........	8.81	6.86	5.21	4.26	2.78	2.85	1.76	2.02	2.66
Value of old pavement..........H.	.76	.18	.72	.78	.14	.48	.12	.10	.10
Total net cost	8.05	6.68	4.46	3.48	2.64	2.07	1.64	1.92	2.56
Life of pavement..L.	20	15	15	12	8	6	4	8	5
Cost per year ....X.	0.402	0.445	0.297	0.290	0.330	0.345	0.410	0.240	0.512

The data assumed in computing this table may be regarded as fair averages for pavements located in cities in the Mississippi Valley, on streets having a traffic of about 100 tons of travel per square yard per day.

The tonnage of travel, (100 tons per sq. yard per day), assumed in this table corresponds to that on a rather heavy travelled residence street, or a business street of medium travel, in cities of 100,000 to 200,000 population, and the result might be entirely different for a street having a larger or a smaller volume of travel. Thus on a street having a travel of only twenty tons per sq. yard per day a first class McAdam road, if kept in proper repair, would probably have a life of fifteen years, and the cost per sq. yd. per year would be very materially reduced.

It is approximately true that, excluding the action of the elements, the life of a pavement is nearly in inverse ratio to the tonnage of travel over it.

Some pavements, notably those of wood, are subject to natural decay and their life is necessarily limited independent of the amount of travel to which they are subjected. Thus an ordinary cedar block pavement will be destroyed by the decay of the wood, regardless of the quantity of travel over it, in from seven to twelve years. The elements, especially those of water and frost have a disintegrating action on nearly all pavements and in time will destroy them. This fact must be considered in predicting the useful life of any pavement.

Such a tabular statement of the estimated cost per annum of constructing and maintaining the several kinds of pavement proposed for any street may be prepared, using the prevailing prices and the best available information as to the probable life of the different pavements under the volume of travel, as well as other conditions to which they will be subjected, and it will be found especially useful in combating the prevalent idea that first cost of the pavement is an important consideration. Thus the table here given shows very clearly that, for a street having similar conditions the pavements of low first cost are in the end the most expensive, even if cost of construction and maintainance only are considered.

POWER REQUIRED TO CONDUCT TRANSPORTATION.

Experiments have been made to determine the tractive power necessary to move given loads over different kinds of roadway surface. It is generally accepted that the power in lbs. required to move a load of 1000 lbs. over the different road surfaces in common use is about as follows :

Over Ordinary earth road	112
Hard clay road	56
Good cobblestone pavement	50
Good Macadam pavement	32
Good granite block pavement	22
Good brick pavement	14
Asphalt pavement	10

It should be explained, however, that no experiments have been made, so far as I know, to determine the resistance to traction of brick pavement, but there is good reason to believe that it occupies a place about midway between granite block and asphalt, and the figure given above is based upon this assumption.

It is customary to assume in treating of the economy of improved pavements, that the total saving of tractive power is in inverse ratio to the resistance to traction of the several road surfaces. This is true only where the loads hauled are all full loads, limited by the maximum tractive power of the horses employed. If we disregard the wear and tear of vehicles and the difference in the expenditure of muscular energy of horses employed in hauling partial loads, it is evident that it will cost no more to haul a half load over granite pavement than over asphalt pavement.

We have no statistics to show what part of the travel over streets consists of full loads. In the censuses of street travel that have been taken, vehicles have been classed with reference to the weight of vehicle and load, and the full loads are not usually separated from the partial loads. A study of such information as we have indicates that it is probably fair to assume that not more than one in twelve of the vehicles passing over the streets of a city are full loads. Since, however, the wear and tear of vehicles and the tax upon the endurance of horses must be considered, we will probably be not far wrong if we assume that one-tenth of the vehicles passing over a street represent full loads. The travel carried by a street is usually estimated in tons of load per day passing a given point, and this is generally reduced to tons per day per foot of width of street, or to tons per day per square yard of pavement ; in the latter case the square yards are computed by dividing the width of street in feet by three.

It is generally accepted that the tractive power exerted by one horse, moving at the rate of two miles per hour, is about 125 pounds.

The power expended in drawing a loaded vehicle over a street surface may be separated into two parts : that required to overcome the axle friction, and the friction between the wheel and the street surface, and that expended in overcoming the resistance due to inequalities in the street surface. From the meagre information we have on the subject, it appears that the first is a very small quantity amounting to not more than ten pounds per ton of load, or five pounds per 1000 lbs. of load. This may be considered a constant quantity independent of the character of the street surface. In exact computations it should be deducted from the total tractive resistance, in comparing one pavement with another, but in the present state of our knowledge and its small relative importance it may be neglected in approximate computations.

Taking the tractive resistance of the several kinds of pavement as given above, and calling the tractive power of a horse 125 pounds, a horse should draw over the various kinds of street surface the following loads:

Over Ordinary earth roads	0.56 tons.
Cobblestone pavement	1.25 "
Macadam pavement	2.00 "
Granite block pavement	2.84 "
Brick pavement	4.46 "
Asphalt pavement	6.2 "

Assuming the cost of a good two-horse team to be $3.00 per day, or one dollar and fifty cents per horse per day, and the distance a load may be carried as twenty miles per day, we may compute the cost per ton-mile on each of these pavements, which will be found to be as follows:

On Ordinary earth roads	26.8 cts.
Cobblestone pavement	12.6 "
Macadam pavement	7.5 "
Granite block pavement	5.3 "
Brick pavement	3.4 "
Asphalt pavement	2.4 "

As the distance traveled in passing over one square yard of pavement is three feet, the cost of hauling loads over one square yard of the several pavements named will be the ton-mile cost given above multiplied by three and divided by 5280, the number of feet in a mile. This we find to be as follows:

Over Ordinary earth roads	.01522 cts.
Cobblestone pavement	.00682 "
Macadam pavement	.00426 "
Granite block pavement	.00301 "
Brick pavement	.00193 "
Asphalt pavement	.00136 "

Having now the ton-mile cost per square yard of transportation over the different kinds of pavement, and the total tonnage per square yard over any street, we may form an equation which will represent the cost per square yard per year that can profitably be invested in a pavement in order that the business may be done most economically.

If T = Total tonnage per square yard per year.

$\frac{T}{10}$ = Full load tonnage per square yard per year.

f = Cost per ton-mile per square yard.

X = Cost of pavement per square yard per year.

$$\text{Then } X = \frac{Tf}{10}. \tag{2}$$

But in the formula for the cost of conducting and maintaining the pavement per square yard per year, we found that:

$$X = \frac{C + M + IL\left(C + \frac{m}{3}\right) - H}{L}$$

Hence we have:

$$\frac{C + M + IL\left(C + \frac{m}{3}\right) - H}{L} = \frac{Tf}{10}. \tag{3}$$

If we reduce and simplify this formula and substitute for L and T their probable value for any street that is to be paved, and the value of f for each variety of pavement, we can determine the kind of pavement most economical to use. In the present state of our information we have not of course the data to enable us to use such a formula, but I present it for the purpose of showing that it should be, and sometime will be, possible to deal with such problems in a scientific manner and thus reach conclusions that will be something more than mere guess work.

In dealing with the cost of transportation over streets the matter of street gradients must not be overlooked. This part of the subject is fully treated in the text-books and I need not go into it here further than to say that when the gradient is considerable it becomes a more important element in the cost of transportation than the co-efficient of tractive resistance; and as in railroad track, the wear of the pavement is greater, and its durability correspondingly less.

That the tractive resistance of pavements is a matter of much greater importance than the public appreciates may be shown by an example.

I will later give the volume of travel on a number of representative city streets. That given as an example of a heavy travelled business street carries a total vehicular tonnage per square yard per day of 243 tons. Counting 300 working days to the year the travel per square yard per year is 72,900 tons, one-tenth of which, 7,290 tons, we may consider full loads. The ton-mile cost per square yard of hauling over granite block pavement we have found to be .00301 cent, and that for brick to be .00193 cent, the difference in favor of brick is .00108 cent. Multiplying 7,290 tons by this difference we have 8.9 cent, which is the saving in cost of hauling, per square yard per year, in favor of brick pavement as against granite block pavement.

In the case of an asphalt pavement we shall find that the saving, figured in the same manner, amounts to 12 cents per square yard per year, equivalent to a sum of about $3,000 per year on a street one mile long. This is, however, only a part of the saving due to smoother pavements, since there are other important elements of economy to be considered, one of which we shall next take up.

WEAR AND TEAR ON VEHICLES.

We have no satisfactory information as to the relative life of vehicles on the different kinds of paved roadway, since no adequate examination has been made into the subject experimentally.

A loaded truly circular wheel rolling over a perfect plane would be subjected to no wear except that due to axle friction and the friction between the wheel and the pavement. As the surface of a street departs from a true plane there will be more or less jolting or concussion which will increase as the unevenness of the street surface increases. When a wheel rolls over a very rough surface rising over an elevated point and dropping to a lower one, the action is similar to that of a weight raised to an equal height and allowed to fall. As is well known, the energy developed by a falling body is as the heighth through which it falls, and we may therefore safely infer that, excluding the wear due to axle friction and attrition between wheel and pavement, the destructive effect of pavements upon vehicles passing over them increases proportionately to the degree of roughness of the pavement surface. Whether the destructive effect increases in direct ratio with the unevenness of the street surface, we have at present no means of knowing, but it seems reasonable to conclude that such is the fact and we shall assume it to be so for the present. This is not true of ordinary earth roads, since the great tractive power required on such roads is largely due to the yielding character of the surface which therefore acts as a cushion to neutralize the effect of impact. If it be accepted as true that the wear and tear of vehicles is proportional to the tractive resistance of pavements we can form some estimate of the relative cost of maintaining vehicles on the several kinds of pavement.

Let us, for the purpose of illustration, assume that on a street whose surface is a perfect plane, a vehicle with perfect wheels is used, and that under the combined action of friction and deterioration it would have a life of fifteen years of constant use ; and that if the effects of friction and deterioration could be eliminated it would be worn out by concussive action due to unevenness of street suface, on the smoothest of all pavements, asphalt, in twelve years. By the combined effects of the two, it would be worn out in six and two-thirds years. If the wear due to the second cause increases in direct ratio with the tractive resistance of the several kinds of pavement it will be found that the vehicle will have a life, if constantly used, on these pavements as follows :

On Asphalt pavement	6.7 years.
Brick pavement	5.1 "
Macadam pavement	3.7 "
Granite block pavement	3.2 "
Cobblestone pavement	1.4 "

At first sight it would seem that these figures must be wrong, but on more careful consideration it seems not unreasonable that a vehicle should last at least four or five times as long on the smooth surface of an asphalt pavement as on the very hard and uneven surface of an ordinary cobblestone pavement.

The average cost of the vehicles used on the city streets is probably not far from $150 each. Neglecting for the present the item of cost of repairs and interest on investment, the first cost of a vehicle, distributed over its life when used constantly on the several pavements named above would be :

	Life.	*Cost per year.*
On Asphalt pavement	6.7	$ 22.39
Brick pavement	5.1	29.41
Macadam pavement	3.7	40.54
Granite block pavement	3.2	46.87
Cobblstone pavement	1.4	107.14

I have at hand the number of vehicles in use in only one city, but it is probable that the relative number in other cities does not very greatly differ from this one.

In Cincinnati, Ohio, where all vehicles must be licensed there are in use 9100 vehicles. The reported length of streets in this city is 500 miles. There are, therefore, about 18 vehicles to each one mile of street.

Knowing the number of vehicles per mile of street, and the cost of these vehicles per year, we can readily compute the relative cost of maintaining vehicles over the several kinds of pavements, and while the figures and deductions here given may be, and probably are not strictly correct, they serve to illustrate the fact that this item is one of such magnitude as to be a very important element in the problem of street paving. It is, however, one to which very little attention has been given.

VOLUME OF TRAVEL.

Some reference has already been made to this division of our subject. Enough has been said to show that the amount of travel over any street it is proposed to pave, is a very important factor in the problem of selecting the kind of pavement to be used.

The statistics of travel over streets are as yet very meagre, but we have the result of censuses of travel, reduced to a common basis on quite a large number of typical streets in American cities. These statistics are exceeding interesting and valuable in studying the various problems connected with street paving. It is only from such statistics that we can determine the durability in use of the several kinds of pavement under different conditions of travel. Excluding natural decay and the destructive action of the elements, it is known that the life of a pavement bears a close relation to the quantity of travel passing over it, and can be more accurately measured by tonnage of travel than by years. A statement that the pavement on a certain street has endured for a given number of years, is of little value unless accompanied by a statement of the tonnage of travel carried during that period. The truth of this is so evident as to need no argument.

Since the life of a pavement is such an important element in any discussion of the paving problem, it is surprising that no adequate efforts are being made by municipal authorities or even by municipal engineers to collect and record the facts by which alone the endurance of a pavement can be determined.

From the statistics available I may give you some facts that will enable you to form an idea of the volume of travel on certain classes of streets, the figures given being in each case the result of a careful census of travel, on a street of the class to which it belongs, in cities of 100,000 and more population.

TABLE B.

Showing Volume of Travel on several streets in American Cities.

OBSERVATIONS PER DAY.	Residence Street, light travel.	Residence Street, medium travel.	Residence Street, heavy travel.	Business Street, light travel.	Business Street, medium travel.	Business Street, heavy travel.
No. of horses passing point of observation	140	644	2007	601	2368	4578
No. of vehicles " " "	92	508	1694	508	1664	3640
Estimated tons of horses	70	322	1003	300	1184	2289
" " vehicles	57	355	970	454	1916	3246
Total estimated, horses and vehicles	127	677	1973	754	3100	5535
Total tons horses and vehicles per sq. yd.	11.7	40.6	168.3	51.3	232.9	415.0
Per ct. of vehicles and loads of 1 ton or less	91.3	89.5	95.5	77.2	62.0	71.6
" " " 1 to 3 tons	8.7	8.7	4.2	19.6	32.5	25.3
" " " over 3 tons	0.0	1.8	0.3	3.2	5.5	3.1
Per ct. of horses walking	48.6	14.3	9.4	41.7	48.6	32.8
" " trotting	51.4	84.2	89.3	58.2	51.3	67.1
" " speeding	0.0	1.5	1.3	0.1	0.1	0.1

If we multiply these quantities of daily travel by the number of working days in the year, say three hundred, we shall have the amount of travel per year over these streets. Thus on the heavy traveled business street the travel, including both horses and vehicles, passing a given point in one year amounts to 1,660,500 tons.

It is doubtful if many people and some engineers have any adequate idea of the enormous volume of business that such a street carries, and the consequent wear and tear to which the pavement is subjected. In studying street travel it is important to know the number of horses as well as of vehicles. In reducing the horses to tonnage it is generally estimated that each horse weighs 1,000 pounds. Observation seems to indicate that, compared ton with ton, horses are more destructive to pavements than vehicles. The sharp blows of their heavily shod feet, particularly if the shoes have sharp steel calks and toes, are very destructive to stone block and brick pavements, as they spall off the corners of the blocks, giving them, in time, the rounded shape familiarly known as "turtle-backed." The velocity of movement of horses and vehicles is an important detail of street census work, since it is fairly well ascertained that the destructive effect of both horses and vehicles increases directly with the speed.

It will be very evident from what has been said that a large amount of work must be done in collecting and collating the statistical and experimental data absolutely required before positive conclusions can be reached and formulated; and it may be appropriate to point out some of the more important of the data required, in order that the subject may be dealt with in an intelligent and satisfactory manner.

First—It is essential that we should have full and exhaustive statistics of street travel. Censuses of travel over a large number of streets of all classes should be secured to determine with close approximation the measure of durability of the principal kinds of street pavement, under all usual conditions. In this way the life of each can be determined in terms of tonnage, which is the only true measure. Censuses of travel should be taken under a uniform system, in uniform units, so that the results from one city may be comparable with those from other cities.

Second—It is important to have the tractive resistance of the several kinds of pavement under the various conditions of actual use accurately determined. While the co-efficients I have used in this lecture are generally accepted as reliable, it is a fact that they were obtained by different observers, using different methods and different apparatus. The velocity of traction was in many cases not accurately recorded, and the condition of the surface of the pavement when the determination was made are mostly unknown. These co-efficients are therefore at least open to question, and certainly do not rest on a scientific basis. This co-efficient has not been determined at all for some kinds of modern pavement. A systematic and scientific determination of this co-efficient made under a uniform system is very desirable.

Third—Another important class of information now entirely wanting, that is necessary for an accurate estimate of the relative economy of pavements is the effect of the several kinds of pavement on durability of vehicles. There will be found many and serious practical difficulties in conducting a satisfactory investigation in this particular field, but these can and doubtless will be overcome.

Fourth—The value of the element of deterioration by natural causes in the several kinds of pavement should be determined with as much accuracy as possible.

Fifth—The relative cost of cleaning the several kinds of pavement under like conditions should be determined. It is well known that it costs more to keep certain kinds of pavement clean than others, but exact figures are not yet available.

Sixth—The cost of maintainance for the several kinds of pavement under various conditions of use should be established. Until these and other like data are obtained it will not be possible to reduce street paving to an exact science. In the meantime engineers must make use of partial information and exercise their best judgment in selecting the kind of pavement that promises the best return for the money invested in it. It is not, however, necessary to treat the subject in the hap-hazard way that has been too common in the past, and is much in evidence at the present time. It should be possible for the engineer to at least classify and assign values in a rude way to the arguments for and against any one kind of pavement, and to weigh the good or bad qualities of one kind against those of other kinds. Without some system by which this may be done it is necessarily difficult, if not impossible, to reach a verdict which shall represent the algebraic sum of the evidence.

I will close with an illustration of how this may be done in a way that ought to prevent any very great mistakes of judgment in selecting a pavement for the average city street.

The *general* heads under which the qualities of a pavement may be divided, are: First, Cost of construction, and maintenance; Second, Economics of use; Third, Effect of the pavement on the public health, which we may call its sanitary qualities; and Fourth, Those qualities that affect the comfort of the people using it, or living along the street.

To each of these four general classes of qualities we may assign a relative weight or value in parts of one hundred.

Of these qualities the first, "Cost of construction and maintenance" is of least importance. It may, in fact be shown, that it is of very little relative importance, and we may assign to it a weight of ten.

The second, "Economy in use" is, when considered from a commercial point of view, the *most* important, and we may assign it a weight of 40.

The third, "Sanitary qualities" is really a very important consideration since nothing can be more important than conditions affecting the health of the community. But street pavements are not usually a very important agency in influencing the health of a city, with the possible exception of decaying wooden pavements, and we may assign a value of 20 to this class of qualities.

The fourth class "Public Comfort" is the one about whose relative importance there is, and is likely to be, most divergence of opinion. On the one hand it may be said that its commercial importance is almost nil since the personal comfort of the business community plays a very small part in business affairs. On the other hand the element of personal comfort is receiving more and more attention in municipal as well as in personal affairs, and I believe justly so. We attach much importance to it in our personal affairs; our clothing, our food and our houses are all designed and regulated by our ideas of taste and comfort, and not with a view to mere utility. The public is constantly demanding greater degrees of comfort in the means of public transportation and every other public function. I think, therefore, we shall not be far out of the way if we assign the number 30 to this class of qualities.

But each of these general qualities, with the exception of the first, is devisable into component qualities, and these component qualities are found in different degrees in the different kinds of pavement. We may represent these qualities by numbers on a scale of 10, the number 10 standing for the highest attainable excellence in that quality, and we may assign to each kind of pavement a number which in our judgment measures the degree to which the quality is found in that pavement. Having done this, we may take the product of the numbers standing for the general qualities and those representing the value of the component qualities. The sum of these products for any one pavement will express the aggregate weight of all the component qualities, and will represent the relative value of the pavement. A table may be arranged which will bring together in convenient form for comparison the numerical values assigned to each kind of pavement, which will show at a glance our estimate of the relative value of each in our judgment.

TABLE C.

Showing Estimated Weight of Qualities of various kinds of Pavement.

	Granite on Concrete.		Asphalt.		Brick on Concrete.		Cedar Block on Concrete.		Macadam.		Cobblestone	
	Coefficient.	Weight.	Coefficient.	Weight.	Coefficient.	Weight.	Coefficient.	Weight.	Coefficient.	Weight.	Coefficient.	Weight.
10. Cost of construction and maintenance	25	25	3.4	34	3.4	34	2.9	29	2.0	20	4.2	42
40. Economy in use:												
Ease of traction	4.6	184	10.0	400	7.1	284	8.0	320	3.1	124	2.0	80
Durability of vehicles	3.2	128	6.7	268	5.1	204	6.0	240	3.7	148	1.4	56
Foot hold for horses	5.5	220	5.0	200	5.0	200	7.0	280	8.0	320	4.0	160
Economy of cleaning	7.0	280	8.0	320	7.5	300	6.0	240	4.0	160	5.0	200
20. Sanitary qualities:												
Freedom from decay	9.0	180	8.5	170	9.0	180	2.0	40	9.0	180	9.0	180
Impermeability	8.0	160	9.0	180	8.0	160	1.0	20	4.0	80	5.0	100
30. Public comfort:												
Noiselessness	3.0	90	8.5	255	6.0	180	9.0	270	8.5	255	2.0	60
Comfort in use	3.0	90	9.0	270	4.0	120	8.0	240	7.0	210	2.0	60
Appearance	5.0	150	8.0	240	7.0	210	4.0	120	5.0	150	3.0	90
Total relative weight	----	1507	----	2337	----	1872	----	1799	----	1647	----	1028

I have thus prepared a table (Table C), showing my own estimate of the general qualities of the several pavements in common use. It, of course, represents my own judgment only. Each individual will have his own opinion of the merits and demerits of different pavements, and will therefore assign different weights to the several qualities. When, however, he has pursued the investigation by this method the totals of the several columns of products will represent in an intelligent and unerring way, the general conclusion to which his opinions in detail lead.

It is not possible in a single lecture to give more than an outline of the subject which I have called the "Economics of street paving." There are many phases of the subject and many interesting details that we cannot touch upon at all in the time available. I have only hoped to call your attention to the importance of the subject, and to enlist your interest in its proper investigation. To those of you who may have the inclination and the opportunity to pursue the study in a systematic and scientific manner, there is ample room in an almost unworked domain. I know of no more promising field in the whole range of municipal economics.

THE HOLDING POWER OF SCREWS IN WOOD.

WALTER J. GRAVES, C.E., 1898.

The first exhaustive investigation of the holding power of ordinary screws of various sizes when inserted under various conditions in different kinds of wood was inaugurated by Mr. Norris M. Works as a thesis subject in 1897. To Mr. Works all credit is due for the inauguration of the test, the devising of the apparatus, and the data obtained by him for white pine. The writer's thesis was but supplementary, continuing the tests for yellow pine and oak. An abstract of Mr. Work's thesis was published in Vol. VI of these Transactions in 1898.

For the purpose of comparison much of the valuable data obtained by Mr. Works is embodied, after slight revision, in this article.

The author used the same series of screws as was employed by Mr. Works. They were the ordinary iron cut screws furnished by the American Screw Co. of Providence, Rhode Island. As the only standard of dimension divulged by the manufacturers was that "the threaded portion is two-thirds the total length of the screw" it was necessary to measure up the series. Table I (revised from Works' thesis) gives the mean of many careful measurements.

The "Effective Length," Column 14, is the length of the threaded portion, measured from the first cut of the tool into the shank to the extreme point, minus a certain number of turns (varying with the different sizes) assumed from inspection as being ineffective in holding power.

The "Areas of Threads" Column 16, are obtained by multiplying the effective length by the outer circumference. This is in accordance with the practice of the Watertown Arsenal.

Description of Woods.—The wood used by Mr. Works in his tests consisted of three strips from three different trees as follows :

Stick A, used in tests of screws which were driven part way with hammer or screwed in to the head, was of very clear straight grained white pine 3″ x 4″ x 8′ long. Specific gravity 0.4186. Rate of growth 9.5 rings per inch.

Stick B, used in comparative preliminary tests was of the best quality of white pine, thoroughly seasoned, of 4″ x 8″ x 10′ long. Specific gravity 0.47. Rate of growth not given.

TABLE I (REVISED) DIMENSIONS OF AMERICAN SCREW CO'S CUT SCREWS.

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
	LENGTHS				DIAMETERS					THREADS				MEAN	AREA
					HEAD	SHANK		THREAD		DOUBLE	TOTAL	PER	EFF	OUTER	OF
NO	TOTAL	HEAD	SHANK	THREAD		GAGE	ACTUAL	OUTER	INNER	DEPTH	NUMBER	INCH	LENGTH	CIRCUM	THREAD
4	1"	06'	.32"	.61"	.22"	.1105"	.110"	106"	.070"	.038"	13.5	22	47"	350"	.166"
8		09	26	64	.32	1631	.161	159	100	059	9.5	15	46	.449	.228
12		12	.15	.68	42	2158	215	213	132	081	70	11	46	669	.306
16		.15	.13	.72	52	2684	267	264	170	.094	7.0	10	.50	830	.411
20		19	.08	74	.62	.3210	319	317	208	.109	6.0	8	52	995	515
				Mean=67											
4	1.5	06	54	90	.22	.1105	.110	108	070	038	20.0	22	76	.350	267
8		09	.46	95	32	.1631	.161	159	100	059	14.0	15	77	.499	.383
12		.12	34	1.04	42	2158	.215	.213	132	081	11.0	11	.81	.669	.543
16		.15	28	1.07	52	2684	267	264	170	094	9.5	9	.82	830	.681
20		.18	.22	1.10	62	3210	319	317	208	109	8.5	8	.85	.995	.845
24		.22	.18	1.10	.72	.3737	374	368	253	115	7.5	7	.84	1.156	.978
				Mean= 1.04											
8	2	.09	.60	1.31	.32	.1631	.161	.159	100	059	19.5	15	1.13	.499	561
12		.12	.54	1.34	.42	.2158	.215	.213	132	.081	14.5	11	1.11	.669	743
16		.15	.54	1.31	52	2684	267	264	170	094	11.5	9	1.06	.830	900
20		.19	.50	1.31	.62	.3210	.319	317	208	109	10.5	8	1.06	.995	1.052
24		.22	.44	1.34	72	3737	.374	368	253	115	9.5	7	1.09	1.156	1.260
				Mean=1.32											
8	2.5	.09	.78	163	32	.1631	161	159	100	059	240	15	145	.499	718
12		.12	87	151?	.42	.2158	215	.213	132	.081	16.5	11	1.39'	669	934
16		.15	.66	1.68	.52	2684	.267	.264	170	094	15.0	9	1.43	.830	1.185'
20		.19	.66	1.65	.62	3210	319	.317	208	109	13.0	8	140	.995	1391
24		22	.64	1.63	.72	3737	374	368	2.53	.115	11.5	7	1.38	1.156	1.590
				Mean=1.62											
12	3	.12	.81	2.07	42	.2158	.213	.213	132	.081	22.0	11	1.84	.669	1.230
16		.15	.92	193	.52	2684	.266	.264	.170	094	17.5	9	1.68	.830	1.395
20		.19	.78	2.03	.62	3210	.318	.317	208	.109	16.0	8	1.78	.995	1.770
24		.22	78	2.00	.72	.3737	.373	368	253	.115	14.0	7	1.74	1.156	2.008
				Mean= 2.00											
12	3.5	.12	1.03	2.35	.42	.2158	215	.213	132	.081	26.0	11	2.12	.669	1.420
16		15	1.00	2.35	52	2684	.268	.264	170	.094	21.0	9	2.10	.830	1.742
20		19	.95	2.36	62	3210	321	317	208	.109	19.0	8	2.11	.995	2.095
24		22	.98	2.30	72	3737	372	363	253	.115	16.5	7	2.04	1.156	2.370
				Mean= 2.34											
12	4	.12	1.25	2.63	.42	2158	.215	.213	132	.081	28.5	11	2.40	.669	1.605
16		15	1.33	2.52	.52	2684	.264	264	.170	.094	22.5	9	2.27	.830	1.885
20		19	1.24	2.57	.62	3210	.319	317	208	.109	20.5	8	2.32	.995	2.301
24		.22	1.04	2.74	.72	.3737	.375	368	253	.115	19.0	7	2.48	1.156	2.870
				Mean= 2.59											
16	4.5	.15	1.60	2.75	.52	2684	.268	.264	.170	.094	24.5	9	2.50	.830	2.071
20		19	1.42	2.80	.62	.3210	.317	.317	.208	.109	23.0	8	2.64	.995	2.625
24		22	1.24	3.06	.72	3737	.377	.368	253	.115	21.0	7	2.80	1.156	3.235
				Mean= 2.93											
20	5	.19	1.62	3.19	.62	3210	.322	.317	208	.109	25.0	8	2.94	.995	2.921
24		22	1.32	3.46	.72	3737	.377	368	253	.115	24.0	7	3.20	1.156	3.710
28		27	1.42	3.31	.82	4263	.426	.421	298	.125	19.5	6	3.06	1.320	4.040
				Mean= 3.32											
20	6	.19	2.05	3.76	.62	3210	.317	.317	208	.109	30.0	8	3.51	.995	3.490
24		22	2.03	3.75	.72	3737	.372	368	253	.115	26.0	7	3.50	1.156	4.040
28		27	1.88	3.85	.82	4263	.426	.421	.298	.125	23.0	6	3.60	1.320	4.751
30		28	1.90	3.82	.92	.4520	.455	.451	.335	.116	23.0	6	3.57	1.440	5.140
				Mean= 3.79								CORNELL	TRANS.	W. J. G

Stick C, used in final tests of holding power of each size of screw was good clear pine, seasoned four years or more, of uniform texture and with a fibre inclined to be rather tougher than in Stick A or B. Specific gravity 0.3828. Rate of growth 15 rings per inch.

The author used woods as follows :—

Stick D, good southern pine, seasoned five years or more, straight coarse grain and clear. Specific gravity .573. Rate of growth, varying with the distance from the heartwood, from 7½ to 10 rings per inch. A narrow streak of dense dark colored heartwood ran down one corner of this stick and in pulling screws parallel with the grain, such screws as chanced to come in this corner gave much higher results.

Stick E, fair quality white oak, well seasoned and with a fine straight grain, very uniform in texture. Specific gravity .729. Rate of growth about 20 rings per inch.

Description of Methods. The screws were inserted with a bit-brace screwdriver in groups along the stick. Shallow ¾″ auger holes were first bored to prevent splitting of the outer fibres. The stick was then clamped to the movable bed of a 50,000 pound Olsen Testing Machine. The screws were clamped, one at a time, around the shank and just below the head by a screw clamp suspended from the upper stationary bed of the machine. The machine was operated by hand at a very slow and uniform rate of speed.

PRELIMINARY TESTS AND INVESTIGATIONS.

Size of Hole for Threaded Portion. This is one of the most important factors in the holding power of a screw. Mr. Works found that a loss resulted from driving screws with a hammer in white pine and that holes of a certain size gave higher results than others. The author continued this investigation and Table II gives a summary of all these tests.

TABLE II.

PROPER SIZE OF HOLE FOR THREADED PORTION.

Ratios of hole diameters to the diameters of inner cylinders of screws.

		White Pine.		Yellow Pine.		White Oak.	
Size Screw.	Inner Diameter.	Perp.	Par.	Perp.	Par.	Perp.	Par.
4″, No. 12	.132″	.95	1.00	.95	(.95) .83	1.07	1.04
3″, No. 16	.170 or .180				(1.00) 1.07		
3.5″, No. 20	.208	.83	.81	.87	.96	.90	.81
6″, No. 30	.335	.84					

NOTE.—The results in parenthesis are as liable to be correct as the stated results owing to the variable texture of the yellow pine.

From a careful study of all the data the following conclusions were drawn :—

1st. The size of the hole should be the same for the same screw both perpendicular and parallel to the grain.

2nd. The smaller the diameter of the screw the smaller should be the ratio of hole to inner cylinder.

3rd. The harder the wood the greater should be the ratio.

4th. Holes should be bored if the maximum holding power is to be realized.

5th. In general the ratio should be from 82% to 97% for white pine, from 90% to 97% for yellow pine, and from 85% to 105% for oak, depending on the diameter of the screw.

6th. The depth of hole should equal the straight threaded length of the screw.

Although this preliminary test was made before the final test for maximum holding power, the conclusions first drawn were somewhat different from the above conclusions and the high or low values for certain screws in Tables IV and V may be due, in part, to improper size of hole as the test was made with holes of size determined by the first and too hasty conclusions.

The following table of sizes would undoubtedly give better results as they conform more closely with the sizes determined by actual test.

The drill sizes vary by sixty-fourths of an inch and can be purchased with square or round shank of any hardware dealer.

TABLE II A.

PROPER SIZES OF DRILLS FOR VARIOUS SIZES OF SCREWS.

		White Pine.			Yellow Pine.			White Oak.		
Screw No.	Inner Diam.	Ratio	Hole	Drill.	Ratio	Hole	Drill.	Ratio	Hole	Drill.
4	.070	1.00	.070	.062	1.00	.070	.062	1.07	.75	.062
8	.100	.98	.090	.094	.97	.097	.094	1.06	.106	.109
12	.132	(.97)	.128	.125	(.95)	.128	.125	(1.05)	.139	.138
16	.170	.93	.158	.156	.93	.158	.156	.97	.165	.156
20	.208	(.82)	.177	.171	(.91)	.187	.187	(.90)	.187	.187
24	.253	.85	.215	.218	.90	.228	.234	.90	.228	.234
28	.296	.85	.237	.234	.90	.266	.265	.90	.266	.265
30	.235	(.84)	.284	.281						

Ratios in parenthesis are from actual tests.
Intermediate ratios can be interpolated.

Size of Hole for Shank.—Tests for the proper sizes of holes for the shank led to the following conclusions:—

1st. For white pine the hole for the shank should be 77% of the diameter of the shank, and in the harder woods 83%.

2nd. The holding power of the shank is so small in comparison with the thread that it makes little difference whether the above percentages are exactly adhered to or not. Calling the ratio 80% for all cases would be sufficiently accurate.

The effect of lubrication, as demonstrated by tests in white pine, is marked, giving a small increase in holding power. The increase would be greater in the harder woods where lubrication becomes an absolute necessity. The best lubricant is the heaviest one which will adhere to the threads. Tar soap gave the best results and was used in these tests.

Effect of Lapse of Time.—From resulting data of this test it is concluded that there is a slight gain in holding power of from 1½% to 1¾% if stress is not applied until some weeks after insertion.

Spacing Required Between Screws.—From a careful inspection of a section cut parallel with the grain it is known that the fibres of the wood are bent upward near the hole and pulled apart and damaged to a distance of three-quarters of an inch each way in white pine and one inch each way in the harder woods.

Although the eye is unable to detect any damage in the cross section cut perpendicular to the grain it is, however, necessary to have ample spacing in this direction because the holding power when placed too near together is lessened.

In white pine the spacing for the larger screws should be one inch in a direction perpendicular to the grain, and two inches in a direction parallel to the grain.

In the harder woods the spacing should be one and one-half inches in the former direction and two and one-half inches in the latter direction. When drawn parallel to the grain the damage to the fibres is to a great extent local and one and one-half inches is ample space between the larger screws.

TABLE III. SUMMARY.

MAXIMUM HOLDING POWER OF SCREWS IN WHITE PINE.

			Perpendicular to Grain.				Parallel to Grain.				
Len.	No.	Thread area.	No. Obs.	Hole.	Av. lbs.	Lbs. per sq. in.	Hole.	No. Obs.	Av. lbs.	Lbs. per sq. in.	Par. / Perp.
1″	4	.166	5	none	194	1170	none	3	113	682	.58
	8	.228	4	"	247	1082	"	3	176	772	.71
	12	.308	4	"	332	1079	"	3	280	910	.84
	16	.411	4	"	430	1047	"	4	242	590	.56
	20	.515	4	"	417	808	.171	3	283	551	.68
1.5″	4	.267	4	"	300	1121	none	4	179	672	.60
	8	.383	5	"	386	1006	"	3	276	715	.72
	12	.543	4	"	569	1050	.138	3	413	773	.73
	16	.681	4	.152	602	885	.149	3	433	637	.74
	20	.845	4	.198	685	812	.171	3	480	701	.70
	24	.978	4	.226	787	805	.219	3	630	645	.80
2″	8	.561	4	none	525	935	none	3	470	838	.89
	12	.743	4	"	792	1066	.138	3	536	721	.67
	16	.900	4	.152	857	955	.149	3	653	725	.76
	20	1.052	4	.198	937	761	.171	3	660	627	.70
	24	1.260	4	.226	1102	877	.219	3	810	643	.74
2.5″	8	.718	3	none	763	1062	none	4	460	640	.60
	12	.934	3	.138	953	1020	.138	3	740	793	.78
	16	1.185	3	.152	1185	1000	.149	3	796	672	.67
	20	1.391	3	.198	1293	930	.171	3	976	755	.76
	24	1.590	3	.226	1383	871	.219	4	1045	657	.75
4″	12	1.230	3	.138	1260	945	.138	3	833	676	.66
	16	1.395	3	.152	1283	920	.149	3	1073	771	.84
	20	1.770	3	.198	1403	794	.171	3	1006	569	.71
	24	2.008	3	.226	1673	835	.219	3	1366	682	.82
3.5″	12	1.420	3	.138	1350	953					
	16	1.742	3	.152	1523	875					
	20	2.095	3	.198	1713	820					
	24	2.370	3	.226	1893	798					
4″	12	1.605	3	.138	1650	1029	.138	3	1056	657	.64
	16	1.885	3	.152	1786	948	.149	3	1310	694	.73
	20	2.301	3	.198	2100	914	.171	3	1533	666	.73
	24	2.870	3	.226	2246	781	.219	3	1663	579	.74
4.5″	16	2.071	3	.152	1720	830					
	20	2.625	5	.198	2084	794					
	24	3.235	3	.226	2473	765					
5″	20	2.921	3	.198	2510	858	.141	3	1960	672	.78
	24	3.710	3	.226	2996	808	.219	7	2206	595	.74
	28	4.040	3	.265	2933	725	.248	6	2190	543	.73
6″	20	3.490	3	.198	2746	786	.141	4	2137	613	.78
	24	4.040	5	.226	3158	782	.219	7	2381	589	.75
	28	4.751	5	.265	3564	765	.248	6	2488	523	.70
	30	5.140	5	.279	3911	760	.279	5	2746	533	.70
		Mean values,				905				670	.74

TABLE IV. SUMMARY.

MAXIMUM HOLDING POWER OF SCREWS IN YELLOW PINE.

			Perpendicular to Grain.					Parallel to Grain.			
Len.	No.	Thread area.	No. Obs.	Hole.	Av. lbs.	Lbs. per sq. in.	Hole.	No. Obs.	Av. lbs.	Lbs. per sq. in.	Par. / Perp.
1″	4	.166	5	.062	294	1770	.062	5	156	941	.53
	8	.228	5	.094	394	1720	.094	5	258	1130	.65
	12	.308	5	.109	380	1230	.109	5	396	1285	1.05
	16	.411	5	.156	518	1260	.156	5	320	780	.62
	20	.515	4	.187	525	1020	.187	5	388	754	.74
1.5″	4	.267	3	.062	443	1660	.062	5	224	839	.51
	8	.383	5	.094	592	1550	.094	5	378	989	.64
	12	.543	5	.125	702	1290	.125	5	556	1025	.80
	16	.681	5	.156	806	1185	.156	5	574	843	.71
	20	.845	4	.187	890	1050	.187	5	686	813	.87
	24	.978	4	.228	977	1000	.234	5	902	921	.92
2″	8	.561	5	.094	780	1390	.094	5	528	941	.68
	12	.743	4	.125	1020	1370	.125	5	740	995	.73
	16	.900	5	.156	1092	1214	.156	5	844	938	.78
	20	1.052	5	.187	1176	1118	.187	5	966	917	.82
	24	1.260	5	.228	1344	1067	.234	5	1180	939	.88
2.5″	8	.718	5	.094	1030	1435	.094	5	780	1085	.75
	12	.934	5	.125	1272	1360	.125	4	903	969	.71
	16	1.185	5	.156	1454	1225	.156	5	1130	955	.78
	20	1.391	4	.187	1633	1175	.187	5	1356	974	.83
	24	1.590	5	.234	1696	1065	.234	5	1312	826	.78
3″	12	1.230	4	.125	1602	1300	.125	5	1076	876	.67
	16	1.395	5	.156	1776	1271	.156	5	1196	858	.67
	20	1.770	5	.187	1896	1071	.187	4	1330	750	.70
	24	2.008	5	.234	2128	1060	.234	5	1750	873	.83
3.5″	12	1.420	4	.125	1805	1270	.125	5	1288	907	.72
	16	1.742	4	.156	2030	1170	.156	5	1764	1013	.86
	20	2.095	5	.182	2648	1264	.199	5	1984	950	.75
	24	2.370	4	.234	2602	1100	.234	5	1902	804	.73
4″	12	1.605	5	.125	2056	1280	.095	5	1560	973	.76
	16	1.885	4	.156	2110	1118	.156	5	1710	909	.82
	20	2.301	4	.187	2418	1049	.187	5	2154	936	.90
	24	2.870	4	.234	2593	904	.234	5	2254	786	.87
4.5″	16	2.071	4	.156	2240	1080	.156	4	1833	885	.82
	20	2.625	3	.187	2667	1015	.187	5	2486	948	.93
	24	3.235	2	.234	2700	836	.234	5	2442	756	.90
5″	20	2.921	6	.187	2845	973	.187	3	2603	891	.92
	24	3.710	3	.234	3507	946	.234	3	3040	820	.87
	28	4.040	3	.266	3660	906	.281	3	2973	736	.81
		Mean values,				1199				911	.78

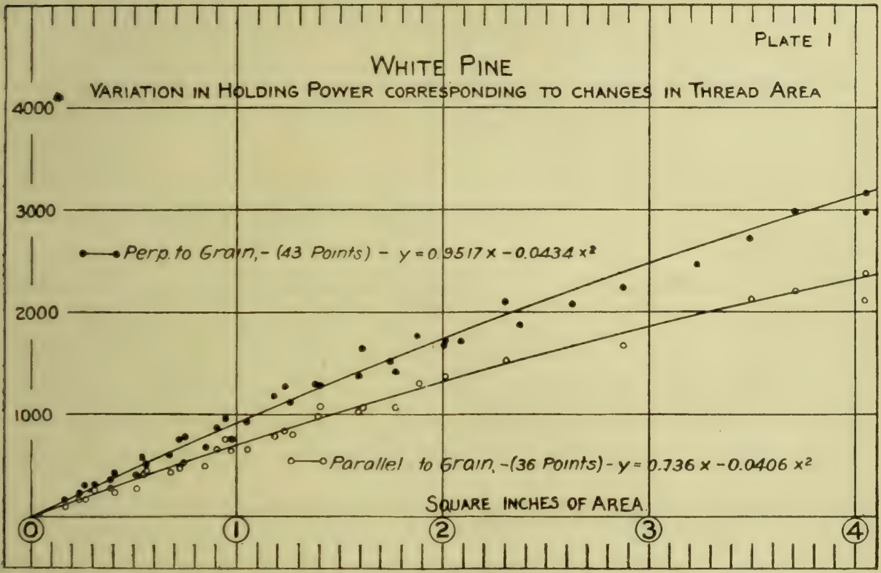
TABLE V. SUMMARY.

MAXIMUM HOLDING POWER OF SCREWS IN WHITE OAK.

			Perpendicular to Grain.				Parallel to Grain.				
Len.	No.	Thread area.	No. Obs.	Hole.	Av. lbs.	Lbs. per sq. in.	Hole.	No. Obs.	Av. lbs.	Lbs per sq. in.	Par. / Perp.
1″	4	.166	5	.062	334	2000	.062	5	225	1450	.73
	8	.228	5	.094	378	1660	.094	5	330	1356	.81
	12	.308	5	.125	500	1623	.125	5	458	1486	.91
	16	.411	5	.156	510	1240	.156	5	524	1273	1.02
	20	.515	5	.187	538	1043	.187	4	382	742	.71
1.5″	4	.267	5	.062	454	1720	.062	5	244	915	.53
	8	.383	5	.094	586	1527	.094	5	402	1050	.69
	12	.543	5	.125	786	1445	.125	5	610	1122	.78
	16	.681	5	.161	810	1190	.161	5	654	960	.81
	20	.845	5	.199	1026	1217	.199	5	770	911	.67
	24	.978	5	.234	1102	1129	.234	5	940	962	.86
2″	8	.561	5	.094	820	1460	.094	5	578	1029	.71
	12	.743	5	.125	1124	1512	.125	4	832	1120	.74
	16	.900	4	.156	1220	1356	.156	5	836	929	.69
	20	1.052	5	.187	1280	1219	.187	5	932	889	.74
	24	1.260	5	.234	1540	1223	.234	5	1078	856	.70
2.5″	8	.718	5	.094	1084	1510	.094	5	752	1044	.69
	12	.934	5	.125	1320	1415	.125	4	905	970	.68
	16	1.185	5	.156	1550	1310	.156	5	998	843	.64
	20	1.391	5	.203	1700	1221	.187	5	1302	936	.77
	24	1.590	5	.234	1834	1153	.234	4	1355	856	.74
3″	12	1.230	5	.125	1662	1350	.125	4	1108	900	.67
	16	1.395	5	.156	1778	1275	.156	5	1268	910	.72
	20	1.770	5	.203	2066	1168	.187	5	1522	861	.74
	24	2.008	5	.234	2476	1233	.234	5	1856	924	.75
3.5″	12	1.420	5	.125	1804	1270	.125	4	1318	927	.73
	16	1.742	5	.156	2182	1252	.156	5	1606	924	.74
	20	2.095	6	.182	2700	1289	.172	3	2120	1010	.78
	24	2.370	3	.234	2860	1207	.234	5	1952	925	.69
4″	12	1.605	4	.141	2140	1338	.125	7	1645	1025	.77
	16	1.885	4	.156	2500	1326	.156	5	1770	940	.71
	20	2.301	4	.203	2863	1240	.187	5	1978	860	.69
	24	2.870	4	.234	3243	1130	.234	5	2394	835	.74
4.5″	16	2.071	4	.156	2618	1262	.156	5	1782	860	.68
	20	2.625	3	.203	3080	1174	.187	3	2263	864	.74
	24	3.235	2	.234	3565	1100	.234	3	2677	829	.75
5″	20	2.921	3	.203	3283	1124	.187	2	2240	768	.68
	24	3.710	3	.234	3577	966	.234	2	2850	768	.80
	28	4.040	2	.281	3897	964	.281	3	3067	759	.79
		Mean values,				1304				964	.74

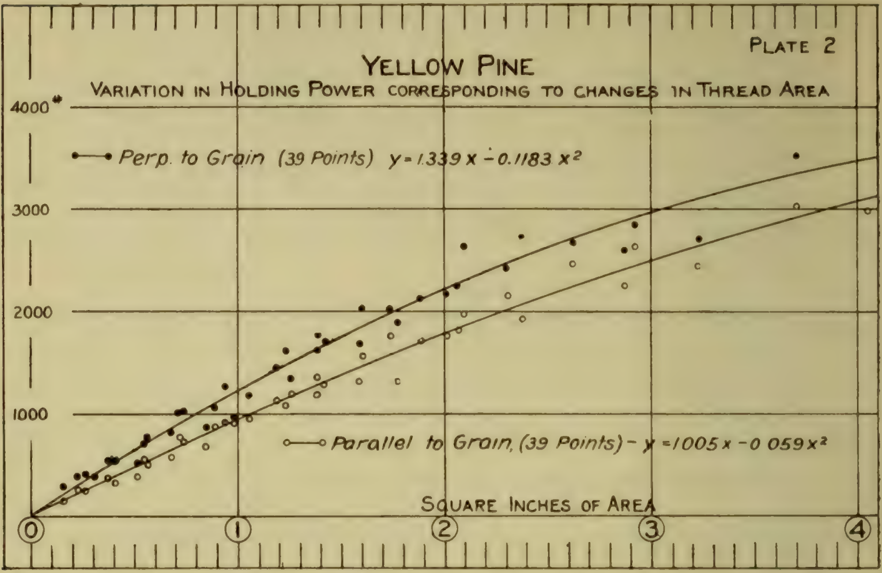
HOLDING POWER OF SCREWS IN WOOD.

Tables III, IV and V give the data of tests from which the accompanying Plates I, II and III were plotted, and from which the curves were computed. The averages for white pine were furnished by Mr. Works and the holding power per square inch of thread area were computed with the areas given in the revised Table I as a basis.

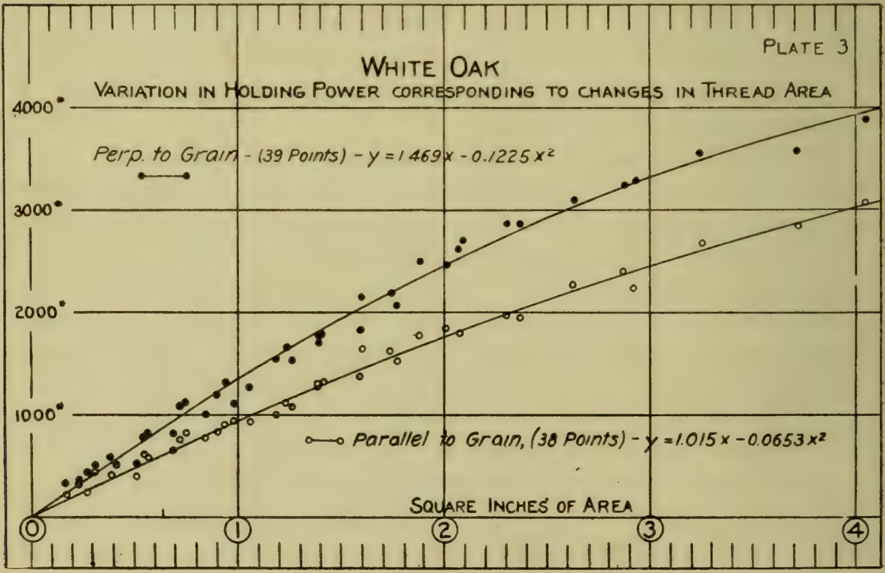


Plates I, II and III show the results of tests graphically and serve to bring out the following facts:

1st. Screws of small diameter hold more per square inch of thread area than those of larger diameter, as the points above the curves are generally for screws of small diameter, while the points below the curves are for screws of large diameter. This fact is better shown by diagrams in the original thesis which are not reproduced here. The writer is unable to assign a reason for this. A trial, using co-efficients depending upon the ratio of diameter of inner cylinder to outer, resulted in bringing points a little nearer the line, but the gain was so small that it was considered an unnecessary refinement. A co-officient based upon the ratio of area of a single thread to the diameter might have given better results. The supposition is that the fibres are not damaged as much in proportion to the thread area by the smaller as by the larger screws; and also, that small screws bend up the fibres resting upon their threads nearly as much as the larger ones.



2nd. There is a diminution of holding power as the length increases, especially for screws longer than four inches, the assumption being that all the threads do not come to an equal bearing, some threads taking



more stress than others ; and also, that the holding power due to friction is less in proportion to the thread area in case of the larger screws.

3rd. The relation between holding power and thread area is expressed by a parabola in each of the six cases. Mr. Works gave a straight line as best fitting the points for white pine. His data, worked up with the thread areas in revised Table I as a basis, give a parabola, although of less curvature than those for the harder woods.

The relation between length, diameter and holding power, was best shown by other diagrams in the original thesis, which also included practical diagrams of the actual holding power for the various sizes of screws and also the relative cost per pound of holding power for the series.

Tests were also made with two sizes of the Rogers drive screw, the results of which are briefly embodied in the final conclusions. These screws are made with a smooth thread of great pitch which enables them to be easily driven with a hammer into hard or soft woods. Their holding power lies between that of the nail and the ordinary screw.

Further investigation might be in the nature of shear and impact tests ; and a test of a complete series of the Rogers drive screw would be of value.

GENERAL CONCLUSIONS.

Wood screws, when drawn parallel to the grain, depend for their resistance upon the shearing strength of the wood parallel to the grain.

When pulled perpendicular to the grain the strength is a function of several conditions : of the shearing strength of the wood ; parallel to the grain, perpendicular to the grain, and across the grain in a plane parallel to the fibres ; also of the flexual stiffness of the fibres.

In order that the maximum holding power may be realized, holes correctly spaced and of the proper diameter and depth must be bored for the thread and shank ; the threads must be well lubricated with soap, or something of that nature, and inserted with a screwdriver—not driven with a hammer.

Driving with a hammer always lessens the holding power because the fibres of the wood or the screws themselves are damaged.

There is a very slight gain in the holding power if stress is not applied until some weeks after insertion.

The increase in holding power due to the insertion of the shank is very small, (about 5 %). On the contrary, the splitting tendency induced is likely to weaken such power.

The holding power is proportional to the area in contact for screws less than four inches in length.

The holding power per pound is greater for the smaller sizes. For the screws of one length it decreases as the diameters increase, and for screws of constant diameter there is a slight increase as the screws lengthen.

The ratio of white pine to yellow pine is 72½ %, of white pine to white oak is 69 %, and of yellow pine to white oak is 93 %. When drawn parallel with the grain screws hold a trifle more in yellow pine than in oak.

The holding power per square inch of thread area is greater for the shorter screws, the diameter remaining constant; and greater for screws of small diameter, the length remaining constant. The relation is expressed in all cases by a parabola of slight curvature.

The proportions of screws are an important factor in their holding power. The limit of economical length is four inches and the economical use of lag screws begin with the four inch.

The 2″, 2½″, and 3″ No. 12 screws are the most economical sizes. The heads and shanks of long screws of small diameter should be increased as the heads pull off before the fibres fail. If the makers could be sure that artisans would use screws properly, and in every case drill a hole for their insertion, a reproportioning of screws might be the outcome.

Many small screws are more economical than a few large ones.

In oak the holding power of the fibres approximates or is greater than, the screws of small diameter.

The dark rings or core (the summer or heartwood) of the yellow pine holds 40 % more than the light colored, sap-wood portions.

One turn out (in the case of longer screws) does not lessen the holding power.

A small crack or check near the screw does not influence the result, while a knot greatly increases the holding power.

There is no particular loss occasioned by removing and reinserting.

The factor of safety need not be large.

The relative cost per square inch of thread area and per pound of actual holding strength reaches a minimum with the medium sized screw.

The Rogers drive screw is about two-thirds as efficient as the ordinary type, and fails by overcoming the frictional resistance to turning. Lubrication greatly lessens their holding power and is unnecessary as they may be driven with ease into the hardest wood. There is an economical limit to their diameters.

THE FLOW OF WATER OVER WEIRS.

GARDNER S. WILLIAMS, M.AM.SOC.C.E.

IN CHARGE OF HYDRAULIC LABORATORY OF CORNELL UNIVERSITY.

The general equation of quantity of water flowing past any given point or through any given section is $Q = A\,v$ where A is the area of the cross section and v the mean velocity per unit of time, and $A = b\,d$ and $v = \sqrt{2\,g\,h}$ where b and d are the breadth and depth of the cross section, h the head to which the mean velocity is due and g the acceleration due to gravity. The equation then becomes $Q = b\,d\,\sqrt{2\,g\,h}$. This equation is of course applicable to the case of a weir when b represents the length of the weir, d the depth of water at the crest, and h the head producing the mean velocity at the weir crest. It is to be noted that h, b and H used below are here entirely different quantities, for as the water approaches the weir the elevation of the surface continually decreases, due to the increase of velocity.

In the common equations of weir discharge the fundamental assumption is made that the water approaches the weir in parallel horizontal layers, and that each layer has at discharge a velocity due to its distance below the plane of still water above the weir, and hence the maximum velocity being at the bottom of the escaping sheet, *i. e.*, next the weir, and the top layer having no head, being therefor stationary, the mean velocity will be that due to $\frac{2}{3}$ of the height of the plane of the surface of still water above the plane of the crest of the weir. On this assumption the above equation of weir discharge would become $Q = b\,d\,\frac{2}{3}\sqrt{2\,g\,H}$.

Practically, it is difficult to measure d, although many of the French hydraulicians have attempted it and used it in their formulas, so that it is customary to measure an H from the crest of the weir to the plane of still water, which equals by the above assumption $\frac{3}{2}\,h$ or $h = \frac{2}{3}\,H$, and this H is also used in place of d. Assuming that H is correctly measured, its introduction in the formula in place of d necessitates a co-efficient to reduce H to d. If this were all that was required of this co-efficient, the law of its variation would be quite definite and our weir formulae much less obscure, but there is an inaccuracy in the fundamental assumption of laminated flow that must also be provided for in a co-efficient. This inaccuracy is due to the fact that the distribution of velocities in the escaping water does not conform to the theory, or, in other words, that the theory is wrong, and the point of maximum velocity may occupy a considerable range of position in the escaping sheet, whence the constant factor $\frac{2}{3}$ disappears and in its place we have a variable factor. The result of all this is that a variable coefficient must be introduced into the equation, which can only be determined by experiment.

A further conception of the discharge of water over the standard sharp crested weir is that all the particles of water in the channel of approach are moving toward the cross section of the stream in the plane of the weir, and wherever the head is measured, it is usual to correct it by computing an additional head from the mean velocity of the approaching water based upon the entire area of the filled cross section of the channel of approach in the plane of measurement of head. Corrections for velocity of approach made in accordance with this conception have been found entirely too small, and the increase required to account for the actual discharge over the weir has a range from 0 to nearly 100% of the computed velocity head. Mr. Francis assumed that the correction for velocity of approach should be based upon a section $b H$ rather than that of the whole filled channel $b (H + P)$ where P is the hight of the crest of the weir above the bottom of the channel of approach. It may be stated without reserve that neither of these assumptions is strictly correct when applied to a sharp crested weir.

The true phenomena of the flow of water over the standard sharp crested weir have never yet been fully investigated. Mr. James B. Francis was the first to notice the ones most intimately connected with velocity of approach. Messrs. Fteley and Stearns added much, in fact most, of our knowledge of the subject, but some investigations by Mr. Emil Kuichling, at Rochester, those of M. Bazin, at Dijon, France, a long series carried on by the writer in the Hydraulic Laboratory of this University, together with a few very interesting ones made by him elsewhere, have been needed to place the subject upon a tangible basis.

Combining the data of these numerous and widely separated experiments it appears that as water flows over a sharp crested weir without end contractions, at low heads there is formed behind (up-stream from) the weir a prism of water in which there is practically no velocity toward the weir. This prism is bounded at its ends by the sides of the canal and has for its faces the bottom of the canal, the plane of the weir or the up-stream face of the weir bulk-head, and a curved surface concave upward extending from the crest of the weir to the bottom of the channel at an as yet undetermined point up-stream. This point appears however to be at a distance from the foot of the weir equal to at least twice its height, and may be quite probably ten or twelve times that distance. Pressures correctly taken within this prism will be found to be always in excess of those taken in the channel outside of it, due probably in the main to the impact of the flowing water against the concave upper surface of the prism, and we may therefore designate it as the prism of pressure.

As the discharge of the weir increases the upper surface of this prism becomes more concave and its intersection with the bottom of the canal approaches the foot of the weir bulk-head. The water within the prism becomes agitated and rolls about in eddies, portions of it escaping now and then over the crest of the weir and other particles entering it from the flowing stream to take their place. Next the foot of the weir bulk-head there exists a high velocity up-stream which is rolled back against the bulk-head at a point about half its height. This current sweeps the bottom of the channel clean next the weir at high discharges and piles up accumulations of sediment and detritus in a heap across the middle of the channel at a short distance from the weir.

Since this condition of affairs exists it follows that so long as the point for measuring the head is not changed the area of the section through which the velocity of approach is to be figured will change as the head changes, the pressure prism cutting out less and less of the area as the head increases. It therefore appears that so far as the bottom conditions along the flowing stream are concerned its cross section increases at points near the weir as the discharge increases, by the lowering of the surface of the prism of pressure as well as by the rise of the upper surface of the water.

Considering now the upper surface of the flowing stream we find the opposite condition to prevail, for as the velocity increases the height of the surface is lowered so that at the crest of the weir the depth of the water may be less than ¾ H. The distance back of the weir to which this surface curve extends has not yet been accurately determined but it evidently depends upon H and P and has been observed to extend up-stream a distance of more than twice the height of the weir.

With weirs having end contractions suppressed it is practically impossible to accurately measure the head in the channel directly. It becomes necessary to use a chamber connected to some form of orifice in the walls of the feeding channel or to some equivalent device. For this purpose four different types of devices have been used, and all introduce some uncertainties.

The first device is that used by Mr. Francis and consisted of a small opening in the side of the channel of approach. This opening must be at right angles to the side of the channel and its edges must be exactly flush.

The second device was added by Mr. Francis as a plane surface set near the middle of the channel parallel to the direction of flow and having similar openings connected by pipes to reservoirs outside of the channel. Messrs. Fteley and Stearns introduced additional openings of this sort on their long weir in order to measure the head simultaneously at various points in the stream.

A third device consists of a pipe about one inch in diameter perforated at frequent intervals and laid a few inches above the bottom of the channel,—the openings being on the under side,—transversely to the direction of flow, or parallel to the crest of the weir.

The fourth device consists of a similar pipe perforated on four sides and set parallel to the direction of flow, or at right angles to the crest of the weir, at any desired distance above the bottom of the channel.

As the presence of high velocity means diminution of pressure head it may be safely assumed that none of these devices will communicate to the connected reservoir a height equal to that in the channel itself unless they connect to the prism of pressure. At first thought this statement may seem unwarranted but we have only to reflect that we may have a velocity in a pipe or open channel so high that no water would escape through openings in the side, whence it follows that the pressure of flowing water against the enclosing walls will vary with the velocity. Since the velocities are not uniformly distributed throughout the cross section of the channel it follows that these devices can only represent the heads in certain parts of it, for the head read from any of them at a place of high velocity must be lower than that read in the same channel where the velocity is less. It becomes at once important to determine where the head should be read. Because of the very irregular eddies in the prism of pressure this becomes an unsatisfactory place to use, and we are then forced to read somewhere in the flowing stream. In order to avoid the pressure prism the reading if taken near the weir should be taken at about the level of its crest. It is also considered desirable to avoid the region of the surface slope and Messrs. Fteley and Stearns have recommended that the head be read about 2½ times the height of the weir up-stream from it. Aside from the errors that may be introduced by reading at a single or a few points it appears so far as our present knowledge goes that either the second or fourth device would give equally satisfactory results, and with short weirs perhaps the first would not be open to serious objection. As to the third device, that of the transverse pipe, it will give low readings of head if set outside of the prism of pressure, the extent of error in the discharge with a two foot head on a six foot high weir, having been found to be over two per cent. as compared with that obtained from a head taken with the first device. The fact that no volumetric determinations of the flow over weirs have yet been made where this device was used, therefore compels its rejection for accurate work except with very low heads.

Thus far we have dealt mainly with the uncertainties and the net result is that the formulae are seen to be incorrectly founded, the corrections for velocity of approach more or less indeterminate, and the position and method of reading the head uncertain. We next come to the certainties, *i. e.*, the experimental determinations of the discharge over weirs where the quantity of water was volumetrically measured.

As to these, it should be remembered that the reliable recorded experiments upon the discharge over weirs with end contractions suppressed, of a size comparable to those usually met with in practice, where the discharge was measured volumetrically, are comprised in three investigations, which cover six different weirs.

The first embraces seventeen experiments by Mr. James B. Francis[1] upon a weir 9.995′ long and 4.60′ high, with heads ranging from 0.73620′ to 1.00600′, and which determines substantially four distinct points on the discharge curve, *viz.*, $h = 0.74'$, 0.81′, 0.98′ and 1.01′.

The second comprises thirty experiments by Messrs. Fteley and Stearns[2], upon a weir 4.999′ long and 3.56′ high, with heads ranging from 0.0746′ to 0.8193′, and ten experiments upon a weir 18.996′ long and 6.55′ high, with heads ranging from 0.4685′ to 1.6038′.

The third is the investigation of Mr. Henry Bazin[3], who made sixty-seven experiments upon a weir 2 meters, 6.56′ long and 3.72′ high, with heads ranging from 0.194′ to 1.012′; thirty-eight experiments upon a weir 1 meter, 3.28′ long and 3.72′ high, with heads ranging from 0.188′ to 1.338′, and forty-eight experiments upon a weir 0.5 meter, 1.64′ long and 3.296′ high, with heads from 0.191′ to 1.779′.

Of these investigators, the Americans read their heads at an opening in the side of the canal, 6 feet up stream from the weir and near the level of the crest, and the openings were in every case less than three-fourths of a square inch in area, but Bazin selected a point 5 meters,

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[1] Lowell Hydraulic Experiments.

[2] Trans. Am. Soc. C. E., Vol. XII.

[3] Annales des Ponts et Chaussees, 1888. Proc. Phila. Eng. Club, 1890.

16.3 feet up stream, and at the bottom of the channel, to read his, and his opening was 4″ in diameter. In each case the openings were at right angles to the direction of the current and flush with the side walls of the canals, except that in some of the Francis and Fteley and Stearns experiments additional openings were made by using plates of wood or metal set parallel to the direction of flow, with orifices in them connecting the pipes to the hook gage pails.

Each of these investigations has produced a formula for weir discharge and from the data of the American experimenters and that of others with much smaller weirs, Mr. Hamilton Smith, Jr.,[1] deduced an additional formula for weirs with end contraction suppressed. It will be seen at once that the total number of experiments in the first two investigations is exceeded by Bazin's first series and nearly equalled by his third, while altogether he presents nearly three times as many determinations as do the other investigators.

Bazin's Formula is a very convenient one to use with such weirs, as the effect of velocity of approach being taken care of by a variable coefficient, no approximations are necessary to provide for it, but the formula gives results somewhat different from those of the other three, and while the Bazin Formula fits the Bazin experiments excellently, it does not fit the others as well.

Considering the positions at which the heads have been read by the Americans and by Bazin, it seems certain that if any difference exists, the velocity will be greater at a point 6 feet from the weir and level with its crest than it will be at a point 10 feet farther up stream and at the bottom of the channel. Since we know that as velocity increases the pressure head decreases, we would expect that heads read at the American position would be lower than those read at Bazin's point, for the same discharge, and hence we should expect that Bazin's Formula would give and his experiments show a less discharge for a given head than would be the case with the others. This appears to be true with a weir 11.24′ high only, with heads above 1.39′. Below that head Bazin's Formula gives a greater discharge than does that of Mr. Francis. Table I shows a comparison of five experiments, on five of the weirs already referred to, in which the measured discharges Q are given and also the computed discharges for each experiment by the four formulæ.

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[1] Hydraulics.

TABLE I.

Experimenter.	l ft.	p ft.	Obs. h	Obs. Q. cu. ft	Q by Formula Cu. ft. per Sec.			
					Francis.	Fteley & Stearns.	Bazin.	Smith.
J. B. Francis	9.995	4.60	0.9760	32.436	32.290	32.300	32.784	32.406
Fteley & Stearns	18.996	6.55	1.4546	112.066	111.890	112.054	112.550	112.240
" "	4.999	3.56	0.8118	12.466	12.307	12.459	12.557	12.497
Bazin	6.56	3.72	0.9794	21.930	21.418	21.478	21.928	21.715
"	1.64	3.30	1.0158	5.754	5.664	5.689	5.753	5.961

From this table it is seen that Bazin's formula gives, in the first three cases, discharges higher than those actually observed, and that the other formulae, except in one case, give for Bazin's experiments discharges less than the observed. Before selecting these experiments, an attempt was made to test the homegeneity of the experiments of each of the observers by computing from the observed Q values of $n^{\frac{2}{3}}h$, which, if n were constant, would give a straight line when plotted as abscissas to the observed h as ordinates. It was found, of course, that n was not constant, but was increasing as the head increased. This plotting, however, demonstrated the consistency of the different experiments of each series with each other. From these observations it appears, therefore, that we have to explain the high values of Q obtained by Bazin, or the low values by the other observers, as theoretically the former should have obtained the low values. A careful study of the conditions of the several experiments leads to the conclusion that the devices used by the Americans for starting and stopping the flow and determining the heights in the measuring basin were capable of more precise operation than were those of Bazin, and hence, patriotism aside, I conclude that the Qs determined by Mr. Francis and by Messrs. Fteley and Stearns were more nearly correct than were those of Bazin, and consequently it follows that the formulae of Francis, Fteley and Stearns and Smith are probably—except perhaps in the last case with short weirs, where experiments upon weirs only eight inches long have been necessarily resorted to in determining the formula,—more accurate than is the recent formula of Bazin. At the same time it appears that even the Francis formula may give low results at high heads, although this is perhaps compensated for in the fact that high heads upon ordinary weirs will probably be accompanied by roughness and turbulence of the approaching water, which quite appreciably reduces the discharge for a given head.

In view of the foregoing it seems doubtful if the data now at hand warrant the presentation of a formula for weir discharge as applicable under general conditions. The best we can do is to reproduce as nearly as possible all the conditions of that one of the several series of experiments which comes nearest to fitting the case in hand and rely rather upon a comparison of the results obtained, with those of the other experimenters ; for the application of any of the weir formulæ to conditions different in any particular from those under which the experiments were made whence they were deduced, is, to say the least, questionable.

It is interesting to note that in the most widely known and generally accepted series of experiments upon the flow of water in pipes, the discharge was measured over a weir and computed by a formula which no one today considers reliable ; that in probably the best published series of modern pipe experiments the discharge was measured over weirs which did not strictly conform to any of the standard precedents, and that the ratings of the larger Venturi meters have been determined by comparison with the flow over a weir upon which it is very doubtful if the head was measured in a manner suitable to the application of any of the weir formulæ. A consideration of some of these points coupled with a realization of the vast financial interests dependent upon the correct measurement of water, may naturally lead us to wonder at the very few thousands of dollars expended upon hydraulic investigation, particularly when we reflect upon the hundreds of thousands invested in astronomical observation, concerning which an eminent astronomer has remarked that if every observatory was wiped out in a single day we would still have an abundance of astronomical knowledge to serve the world for all practical purposes for the next one thousand years.

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